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Project No. ACA11-007-00
March 27, 2012

Mr. Pete Anaya, P.E., Director of Engineering Services
City of Corpus Christi
Engineering Services
1201 Leopard Street
Corpus Christi, Texas 78401-2825

Re: Geotechnical Engineering Services
Park Road 22 Bridge Project
Along South Padre Island Drive
Approximately 3,000 ft South of its Intersection of Commodores Drive
Corpus Christi, Nueces County, Texas
City of Corpus Christi Project No. 6281

Dear Mr. Anaya:

Raba Kistner Consultants, Inc. (RKCI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with **RKCI** Proposal No. PCA11-010-00, dated August 29, 2011. Written authorization to proceed with this study was received by our office via electronic-mail attachment on Friday, November 18, 2011, by means of an executed contract between the City of Corpus Christi (CLIENT) and RKCI titled “Agreement for Geotechnical Services”, dated November 3, 2011, which includes our proposal document as Exhibit “A” Scope of Work. Subsequently, an original of the executed contract document was received by our office via U.S. mail.

Please note that the drilling operations for this project were delayed until the orientation of the proposed water exchange channel and the location of the arch bridge structures were determined by the project design team. Subsequently, the boring locations were staked in the field by Urban Engineering, the project’s consulting engineering firm, on Friday, January 20, 2012. The submittal of this report was pending until the cross-section of the proposed arch bridge structures and the anticipated traffic loading information for the proposed reconstruction of the existing road alignment were provided to us on Monday, March 12, 2012 and Wednesday, March 14, 2012, by the project’s consulting engineering firm and the Texas Department of Transportation (TxDOT), respectively.

The purpose of this study was to drill borings within the subject sites and to develop information concerning foundation recommendations for the proposed arch bridge structures, associated retaining wall structures of the water exchange channel, and a lift station structure; as well as to provide pavement recommendations for the proposed reconstruction of the existing road alignment.

The following report contains our foundation and pavement recommendations and considerations based on our current understanding of the proposed elevations of the subject structures, design tolerances, and structural and pavement loads. If any of these parameters change, then there may be alternatives for value engineering of the foundation and pavement systems, and **RKCI** recommends that a meeting be held with the City of Corpus Christi and the design team to evaluate these alternatives.

We appreciate the opportunity to be of professional service to you on this project. Should you have any questions about the information presented in this report, please call. We look forward to assisting City of Corpus Christi during the construction of the project by conducting the construction materials engineering and testing services (quality assurance program).

Very truly yours,

RABA KISTNER CONSULTANTS, INC.

STATE OF TEXAS
KATRIN M. LEONARD
95810
LICENSED
PROFESSIONAL ENGINEER
March 27, 2012

Katrin M. Leonard, P.E.
Manager, Engineering Services

Martin Vila, P.E., F. ASCE
Senior Vice President

Attachments

KML/MV

Copies Submitted: Above (1)
Urban Engineering (1)
Tricone Engineering (1)

GEOTECHNICAL ENGINEERING STUDY

For

PARK ROAD 22 BRIDGE PROJECT
ALONG SOUTH PADRE ISLAND DRIVE
APPROXIMATELY 3,000 FT SOUTH OF ITS INTERSECTION OF COMMODORES DRIVE
CORPUS CHRISTI, NUECES COUNTY, TEXAS
CITY OF CORPUS CHRISTI PROJECT NO. 6281

Prepared for

CITY OF CORPUS CHRISTI
Corpus Christi, Texas

Prepared by

RABA KISTNER CONSULTANTS, INC.
Corpus Christi, Texas

PROJECT NO. ACA11-007-00

March 26, 2012

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Important Information About Your Geotechnical Engineering Report

INTRODUCTION

Raba Kistner Consultants, Inc. (RKCI) has completed the authorized subsurface exploration and foundation and pavement recommendations for the proposed Park Road 22 Bridge Project to be located along the Park Road 22 (South Padre Island Drive), approximately 3,000 ft south of its intersection with Commodores Drive in Corpus Christi, Nueces County, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation and pavement design and construction considerations.

PROJECT DESCRIPTION

It is our understanding that the proposed project consists of the design and construction of two precast concrete arch bridge structures to be located along the existing alignment of Park Road 22 (South Padre Island Drive), approximately 3,000 ft south of its intersection with Commodores Drive in Corpus Christi, Nueces County, Texas. We understand the arch bridges will include pedestrian traffic crossings on both ends in a west-to-east direction off Park Road 22 and they will support vehicular traffic over a channel which is anticipated to promote water exchange between the east and west residential subdivisions off Park Road 22. We understand that the materials to be excavated/dredged in order to accommodate the water exchange channel are preferred to be used as general fill materials for the construction of its retaining walls. We further understand that the project will also include the design and construction of an approximately 20-ft deep lift station to be located along the west side of Park Road 22, approximately 1,400 ft south of its intersection with Commodores Drive.

On the basis of the plans titled "Conceptual Layout for Park Road 22 Bridge (Bond 2004)", dated February 2012 (Sheets 1 through 6), provided to us via electronic-mail attachment on Monday, March 12, 2012 by Mr. Rhodes "Chip" Urban, P.E., R.P.L.S., Principal, with Urban Engineering, the project's consulting engineering firm, we understand that the earthen ramps associated with the proposed arch bridge structures will be about 1,600-ft long, about 200-ft wide, and about 14-ft high at its midpoint. Based on the same drawings, we understand that each of the two precast concrete arch structures will be about 42-ft wide, with a 15-ft wide sidewalk (proposed elevation +4 ft) and an about 25-ft wide water exchange channel with about 7-ft of standing water (proposed bottom of channel elevation -6 ft). Sheetpiling is proposed to be installed at the interface of the water exchange channel and the sidewalk at each end of the arch structures down to a conceptual elevation of about -16 ft. (In general, for this type of cantilever sheetpile design, the sheetpiling depth is typically twice as deep as the exposed height of the sheetpile structure.) In addition, on the basis of the site plan titled "Bore Hole Locations – Park Road 22 Bridge (Bond 2004)", dated April 21, 2011, provided to us via electronic-mail attachment on Wednesday, March 14, 2012 by the project's consulting engineering firm, we understand that the ground surface elevations existing at the time of our study at the boring locations range from about 2.0 ft to 7.0 ft above mean sea level (MSL).

Further, based on the traffic information provided to us via electronic-mail transmittal on Wednesday, March 14, 2012 by Mr. Anthony Villarreal, P.E., with the Texas Department of Transportation (TxDOT), we understand that an average daily traffic (ADT) of 17,054 was estimated for 2011, and an ADT of 24,201 is projected for 2031, with 19.7% truck ADT, which equates to about 6,412,000 Equivalent Single Axle Loads (ESAL's) for a 20-year design period.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of South Texas for the use of the City of Corpus Christi and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses and is not intended for use in determining construction means and methods.

The recommendations submitted in this report are based on the data obtained from 17 borings drilled within the subject sites and our understanding of the project information provided to us by others. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the subject sites. The nature and extent of variations across the subject sites may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the sites. No environmental opinions are presented in this report. **RKCI**'s scope of work does not include the investigation, detection, or design related to the prevention of any biological pollutants. The term “biological pollutants” includes, but is not limited to, mold, fungi, spores, bacteria, and viruses, and the byproduct of any such biological organisms.

If final grade elevations are significantly different from the proposed site grading information provided to us by others, our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the subject sites were evaluated by 17 borings, as summarized in the following table.

Proposed Structure	Number of Borings	Depth, ft. *	Depth, ft. *
Arch Bridges	2	80	B-1 and B-3
	2	60	B-2 and B-4
Road Alignment	6	15	B-5 through B-10
Lift Station	1	35	B-11
Water Exchange Channel (Borrow Area)	6	30	B-12 through B-17

* below the ground/pavement surface elevation existing at the time of our study.

The borings (designated as "B-") were drilled on January 23 and from January 26 through February 1, 2012, at the locations shown on the Boring Location Maps, Figures A-1 and B-1. The boring locations were located in the field by representatives of the project's consulting engineering firm, based on a site plan titled "Schlitterbahn Site Plan", dated January 11, 2012, provided to us via electronic-mail attachment on January 11, 2012 by the project's consulting engineering firm.

The borings were drilled to the depths indicated in the previous table using a truck-mounted, rotary-drilling rig. The borings were conducted utilizing hollow stem augers in combination with mud rotary drilling techniques (for some borings) and were backfilled with the auger cuttings following completion of the drilling operations. During the drilling operations, the samples tabulated in the following table were collected:

Sample Type	Number Collected
Split-Spoon (with Standard Penetration Test, SPT)	46
Shelby Tubes (ST)	8
Grab Samples	94

With the exception of the grab samples associated with the arch bridge and water channel exchange borings (Borings B-1 through B-4 and B-12 through B-17), samples were obtained using conventional split-spoon and/or Shelby tube sampling techniques in general accordance with applicable American Society for Testing and Materials (ASTM) standards. Subsurface information for the arch bridge and water exchange channel borings was obtained by using Texas Cone Penetration (TCP) techniques in general accordance with the TxDOT Geotechnical Manual, dated August 2006.

The SPT and ST samples were obtained in accordance with accepted standard practices and the penetration test results are presented as "blows per foot" on the boring logs. Representative portions of the samples were sealed in containers to reduce moisture loss, labeled, packaged, and transported to our laboratory for subsequent testing and classification.

In the laboratory, each sample was evaluated and visually classified by a member of our Geotechnical Engineering staff in general accordance with the Unified Soil Classification System (USCS). The geotechnical engineering properties of the strata were evaluated by the laboratory tests tabulated in the following table.

Test Type	Number Conducted
Natural Moisture Content	148
Atterberg Limits	40
Percent Passing a No. 200 Sieve	46
Unconfined Compressive Strength	2
Dry Unit Weight Determination	7
Sieve Analyses	10
Consolidation	2
Consolidated-Undrained (CU) Triaxial	2
California Bearing Ratio (CBR)	1

The borings conducted using TCP techniques in general accordance with the TxDOT Geotechnical Manual, dated August 2006 (Borings B-1 through B-4 and B-12 through B-17) and their associated laboratory test results are presented in Appendix A. With the exception of the laboratory sieve analyses, consolidations, CU Triaxials, and CBR test results, the results of the field and laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures A-2 through A-11. The results of the laboratory sieve analyses are presented on Figures A-12 and A-13. One laboratory CBR test was performed by combining the bulk soil surficial samples collected from borings conducted along the existing street alignment. This test was conducted to provide information regarding inundated strength, deflection, and swell characteristics of the subgrade soils for pavement analyses. The subgrade laboratory CBR value was evaluated using ASTM D1557 Standard Proctor compaction testing procedures. The laboratory CBR test results are presented on Figure A-14. The laboratory consolidation test results are presented on Figures A-15 and A-16. The laboratory CU Triaxial test results are presented on Figures A-17 through A-20.

Similarly, the borings conducted using conventional split-spoon and/or Shelby tube sampling techniques in general accordance with applicable ASTM standards (Borings B-5 through B-11) and their associated laboratory test results are presented in Appendix B. The results of the field and laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures B-2 through B-8. A key to the classification of terms and symbols used on the logs is presented on Figure B-9. The results of the laboratory and field testing are also tabulated on Figure B-10 for ease of reference.

TCP results are noted as “blows per 6-inch interval” on Figures A-2 through A-11, where “blows per 6 inch” refers to the number of blows by a falling 170-lb (pound) hammer required for 1 ft of penetration into the subsurface materials. Similarly, SPT results are noted as “blows per ft” on Figures B-2 through B-8 and on Figure B-10, where “blows per ft” refers to the number of blows by a falling 140-lb (pound) hammer required for 1 ft of penetration into the subsurface materials. Where hard or dense materials were encountered, the tests were terminated at 100 and 50 blows even if one foot of penetration had not been achieved, for TCP and SPT tests, respectively.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the written request of the CLIENT.

GENERAL SITE CONDITIONS

SITE DESCRIPTION

The subject site for the proposed Park Road 22 Bridge structure is located along Park Road 22, approximately 3,000 ft south of its intersection with Commodores Drive in Corpus Christi, Nueces County, Texas. The site for the proposed water exchange channel is located on both sides of the proposed arch bridge structures, extending to the gulf bay on the east side, and meandering along the existing golf course’s artificial lakes to the west, then curving south to connect to an existing water channel structure. The site of the proposed lift station is located along the west side of Park Road 22, approximately 1,400 ft south of its intersection with Commodores Drive. At the time of our field activities, the project site for the proposed arch bridge can be described as an asphalt-paved road, while the other sites can be described as open, undeveloped, grass-covered tracts of land. In general, the topography at the subject sites is relatively flat, with an estimated vertical relief of about 4 ft across the site. Surface drainage is estimated to be poor.

SITE GEOLOGY

A cursory review of the Geologic Atlas of Texas, (Corpus Christi Sheet, dated 1975), published by the Bureau of Economic Geology at The University of Texas at Austin, indicates that the subject sites appear to be located within a boundary between the Fill and Spoil Formation and Barrier Island Deposits. The fill and spoil formation consists, in part, of fill material dredged for raising land surface above alluvium and barrier island deposits and for creating land. The barrier island deposits consist of sand, silt, and clay deposits (mostly sand) of the Quaternary epoch (Recent Period).

According to the Soil Survey of Nueces County, Texas, published by the United States Department of Agriculture - Soil Conservation Service, in cooperation with the Texas Agricultural Experiment Station, the project sites appear to be located within the Galveston-Mustang-Tidal flats soil association consisting of coastal sands generally above water, and low flats that are flooded daily. The corresponding soil symbols appear to be Mu and Ua, Mustang fine sand and Urban land, respectively.

SEISMIC COEFFICIENTS

Based upon a review of Section 1613 *Earthquake Loads* of the 2009 International Building Code (IBC), the following information has been summarized for seismic considerations associated with this site based on Latitude 27.610468°N and Longitude 97.221864°W for the proposed arch bridge and Latitude 27.610961°N and Longitude 97.222368°W for the proposed lift station.

- Site Class Definition (Table 1613.5.2): **Class E**. Based on the borings conducted for this investigation, the upper 100 feet of soil may be characterized as a soft soil profile.
- Mapped Maximum Considered Earthquake Ground Motion for a 0.2 sec., Spectral Response Acceleration (Figure 1613.5(1)): $\mathbf{S_s = 0.078g}$. Note that the value taken from Figure 1613.5(1) is based on Site Class B and is adjusted as per 1613.5.3 below.
- Mapped Maximum Considered Earthquake Ground Motion for a 1 sec., Spectral Response Acceleration (Figure 1613.5(2)): $\mathbf{S_1 = 0.019g}$. Note that the value taken from Figure 1613.5(2) is based on Site Class B and is adjusted as per 1613.5.3 below.
- Value of Site Coefficient (Table 1613.5.3 (1)): *from worksheet* $\mathbf{F_a = 2.5}$.
- Value of Site Coefficient (Table 1613.5.3 (2)): *from worksheet* $\mathbf{F_v = 3.5}$.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec., adjusted based on equation 16-37: *from worksheet* $\mathbf{S_{ms} = 0.194g}$.
- 1 sec., adjusted based on equation 16-38: *from worksheet* $\mathbf{S_{m1} = 0.068g}$.

The Design Spectral Response Acceleration Parameters are as follows:

- 0.2 sec., based on equation 16-39: *from worksheet* $\mathbf{S_{DS} = 0.129g}$.
- 1 sec., based on equation 16-40: *from worksheet* $\mathbf{S_{D1} = 0.045g}$.

Based on the parameters listed previously, the critical nature of the structures, Tables 1616.3(1) and 1616.3(2), and calculations performed using a Java program titled, "Seismic Hazard Curves and Uniform Hazard Response Spectra" published by the United States Geological Survey (USGS), the Seismic Design Category for both short period and 1 second response accelerations is **IIA**.

STRATIGRAPHY

The subsurface stratigraphy at these sites can be described by three generalized strata. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. For purposes of this report, we have designated the subsurface strata as Strata I through Strata III. The lines designating the interfaces between strata on the boring logs represent approximate boundaries. Transitions between strata may be gradual.

A single core was obtained from the road alignment connecting the Park Road 22 southbound to northbound lanes (located just north of Boring B-7). The existing hot-mix asphaltic concrete (HMAC) thickness was measured to be about 2-1/2 inches, while the flexible base material (FBM) thickness underlying the HMAC was measured to be about 11-1/4 inches.

Stratum I consists of dark grayish-brown to grayish-brown to light grayish-brown to gray to dark brown to brown to light brown to bluish-brown, very loose to very dense, poorly-graded sand soils, poorly-graded sand soils with silt, silty sand soils, and clayey sand soils with seaweeds, shell fragments, and gypsum nodules. This layer was noted in Boring B-1 from the ground surface elevations existing at the time of our study extending down to a depth of about 42 ft, and then again from a depth of about 47 ft extending down to a depth of about 57 ft. In Borings B-2, and B-5 through B-17, this layer was noted from the ground surface elevations existing at time of our study extending down to at least the termination depths of these borings. In Borings B-3 and B-4, this layer was noted from the ground surface elevations existing at the time of our study extending down to depths of about 60 ft and 55 ft, respectively. Moisture contents were measured to range from about 8 to 49 percent for this layer. This stratum is classified as non-plastic to plastic, with the majority of the plasticity indices that could not be determined and with six measured plasticity indices ranging from 7 to 24 percent. Percent passing a No. 200 sieve tests demonstrate percent fines ranging from about 1 to 47 percent for this layer. A single undrained shear strength value of about 0.3 tons per square foot (tsf) was measured, based on a single unconfined compression strength test. Two unit dry weight values of about 96 and 97 pounds per cubic foot (pcf) were measured for this layer. TCP values ranging from 3 blows to more than 100 blows per foot of penetration were measured for this stratum. SPT N-values ranging from 4 blows to 32 blows per foot of penetration were measured for this stratum. These soils are classified as SP soils, SP-SM soils, SM soils, and/or SC soils in general accordance with the USCS.

Stratum II consists of bluish-brown, very stiff, sandy lean clay soils. This layer was only noted in Boring B-1 between the approximate depths of about 42 to 47 ft below the ground surface elevation existing at the time of our study. A single moisture content was measured to be about 26 percent for this layer. This stratum is classified as moderately plastic, with a single measured plasticity index of 15 percent. A single percent passing a No. 200 sieve test demonstrates percent fines of about 64 percent for this layer. A single unit dry weight value of about 96 pcf was measured for this layer. A single TCP value of 79 blows per foot of penetration was measured for this stratum. These soils are classified as CL soils in general accordance with the USCS.

Stratum III consists of grayish-brown to light grayish-brown to bluish-brown, stiff to very hard, lean clay soils, sandy fat clay soils, fat clay soils with sand, and fat clay soils with orange ferrous stains. This layer was only noted in Borings B-1, B-3, and B-4 from beneath Strata I and II soils extending down to at least the termination depths of these borings. Moisture contents were measured to range from about 21 to 39 percent for this layer. This stratum is classified as moderately plastic to highly plastic, with measured plasticity indices ranging from 18 to 77 percent. Percent passing a No. 200 sieve tests demonstrate percent fines ranging from about 64 to 87 percent for this layer. A single undrained shear strength value of about 0.5 tsf was measured, based on a single unconfined compression strength test. Unit dry weight values ranging from about 84 to 94 pcf were measured for this layer. TCP values ranging from 23 blows to more than 100 blows per foot of penetration were measured for this stratum. These soils are classified as CL soils and/or CH soils in general accordance with the USCS.

GROUNDWATER

Groundwater was encountered in the borings during the drilling operations as tabulated in the following table and as described in the drilling logs and the logs of borings:

Boring Identification	Approximate Ground Surface Elevation Existing at the Time of our Study (ft)	Approximate Depth of Groundwater Encountered During the Drilling Operations (ft) *	Approximate Elevation of Groundwater Encountered During the Drilling Operations (ft) **
B-1	2.26	2.5	-0.24
B-2	2.62	2.0	0.62
B-3	2.49	2.0	0.49
B-4	2.53	2.5	0.03
B-5	2.09	2.5	-0.41
B-6	2.22	3.0	-0.78
B-7	2.50	2.5	0.00
B-8	2.14	2.5	-0.36
B-9	1.71	2.5	-0.79
B-10	1.94	2.5	-0.56
B-11	3.95	2.5	1.45
B-12	3.48	1.5	1.98
B-13	2.40	1.5	0.90
B-14	4.89	2.5	2.39
B-15	3.86	2.5	1.36
B-16	4.37	2.0	2.37
B-17	6.99	2.0	4.99

* below the ground surface elevations existing at the time of our study.
**(+) above MSL and (-) below MSL.

It is possible for groundwater to exist beneath these sites at shallower depths on a transient basis following periods of precipitation. Fluctuations in groundwater levels occur due to variations in rainfall, surface water run-off, and the sea level. The construction process itself may also cause variations in the groundwater level.

Based on the findings in the borings and on our experience in this region, we believe that groundwater seepage will be encountered during construction activities. Dewatering methods, such as deep wells and/or well point systems as well as temporary earthen berms and conventional sump-and-pump dewatering methods will be required for the control of groundwater seepage. The dewatering system should maintain the groundwater level at least 5 feet below the level of excavation throughout the period of excavation and construction. Excavations below the groundwater table, generally require lowering the piezometric level to permit construction in a relatively dry state. This should be performed to control seepage into the excavations and to reduce artesian water pressures below the bottom of the excavations.

Wellpoints, deep wells, and/or submersible pump and sump dewatering systems are commonly used. The implementation of these methods should be anticipated for construction of these WTP structure additions. The design of dewatering systems is beyond the scope of this study. Proper construction procedures and equipment will be critical for proper performance of the dewatered foundation excavations. Additionally, protection of personnel entering the excavations and providing a dry, stable subgrade on which to construct foundations will be crucial. For deep foundation excavations, this will include the use of slurry drilling and/or temporary casing (including overdrive techniques) to reduce groundwater seepage and sloughing of the subsurface soils.

FOUNDATION ANALYSES

EXPANSIVE, SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at these sites were estimated for slab-on-grade construction using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). Negligible PVR values were estimated for the stratigraphic conditions encountered in the borings. The PVR value was estimated using a surcharge load of 1 pound per square inch (psi) for the concrete slab and dry moisture conditions within the regional zone of seasonal moisture variation.

The TxDOT method of estimating expansive, soil-related movements is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content. If desired, other methods of estimating expansive, soil-related movements are available, such as estimations based on swell tests and/or soil-suction analyses. However, the performance of these tests and the detailed analysis of expansive, soil-related movements were beyond the scope of the current study. It should also be noted that actual movements can exceed the estimated PVR values due to isolated changes in moisture content (such as due to leaks, landscape watering....) or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

SETTLEMENTS

The subject site for the proposed arch bridge structures is underlain by non-plastic, sand soils that are wet and compressible. High moisture contents and relatively low TCP values were observed in the borings conducted for these borings from the ground surface elevations existing at the time of our study extending down to depths of about 35 to 40 ft. Considering the proposed maximum 14-ft of surcharge, traffic loadings, and structural components of the arch bridges, based on the low shear strengths and high moisture contents of these soils, the sand soils have the potential for settlements under the applied structural loads. The potential settlements can only be estimated once the site grading plans, foundation dimensions, and structural loads have been established for this project.

Drainage Considerations Considerations of surface and subsurface drainage may be crucial to construction and adequate foundation performance of the soil-supported structures. Several surface and subsurface drainage design features and construction precautions can be used to limit problems associated with fill moisture. These features and precautions may include, but are not limited to, the following:

- Installing berms or swales on the uphill side of the construction areas to divert surface runoff away from the excavation/fill areas during construction;
- Sloping of the top of the subgrades with a minimum downward slope of 1.5 percent out to the base of a dewatering trench located beyond the structures' perimeters;
- Sloping the surface of the fill during construction to promote runoff of rain water to drainage features until the final lift is placed;
- Sloping of a final, well-maintained, impervious clay or pavement surface (downward away from the proposed structures) over the select fill material and any perimeter drain extending beyond the building lines, with a minimum gradient of 6 in. in 5 ft;
- Constructing final surface drainage patterns to prevent ponding and limit surface water infiltration at and around the structures' perimeters; and
- Locating the water-bearing utilities, roof drainage outlets, and irrigation spray heads outside of the select fill and perimeter drain boundaries.

Details relative to the extent and implementation of these considerations must be evaluated on a project-specific basis by all members of the project design team. Many variables that influence fill drainage considerations may depend on factors that are not fully developed in the early stages of design. For this reason, drainage of the fill should be given consideration at the earliest possible stages of the project.

FOUNDATION RECOMMENDATIONS

SITE GRADING

Site grading plans can result in changes in almost all aspects of foundation recommendations. We have prepared the foundation recommendations based on the site grading information provided to us by others and the stratigraphic conditions encountered at the time of our study. If site grading plans differ from the information provided to us by others, we must be retained to review the site grading plans prior to bidding the project for construction. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

DEEP FOUNDATIONS – PROPOSED PARK ROAD 22 BRIDGE

We have computed allowable downward vertical capacities for various diameter drilled, straight-shaft piers for the proposed Park Road 22 arch bridge structures. Pier capacity curves were developed using the TxDOT Geotechnical Manual dated August 2006. The results are presented graphically in Appendix C (Figures C-1 through C-8). The capacity curves are based on the TCP blow count data obtained from our borings. The indicated capacities on these figures are for dead plus live loads and take into account skin friction and end bearing. Dead loads should not exceed two-thirds of the computed capacities. Side shear resistance was neglected to the elevation indicated in each of the notes of the drilled pier capacity curves, as well as the bottom one shaft diameter. We recommend that any fill material encountered during drilled pier construction be neglected for side shear resistance.

Chapter 5, Section 3 of the TxDOT Geotechnical Manual (August 2006) provides a table of Maximum Allowable Drilled Shaft Service Loads (page 5-11). This table lists the maximum recommended structural loads for drilled shafts. We recommend that the design team review and consider this information in the event that it is applicable to this project.

Post-construction settlement will be dependent on the final structural loading, pier size and spacing, and group size determined by the design team. We recommend that **RKCI** be retained to review the final loads and pier group layouts, to review pier capacities and to check estimated foundation settlements.

Our arch bridge foundation design recommendations are based on the ground surface elevations existing at the time of our study and the proposed grades shown on the bridge layout, both provided to us by the project's consulting engineering firm via electronic-mail attachment on Monday, March 12, 2012. If final ground elevations change significantly from those provided to us, supplemental recommendations may be required.

The surficial soils encountered in the borings exhibit relatively low shrink/swell potential within the active zone. Therefore, potential uplift forces acting on the pier shafts due to expansive, soil-related movements are negligible at this site.

Allowable Uplift Resistance

Resistance to uplift forces exerted on the drilled, straight-shaft piers will be provided by the sustained compressive axial force (dead load) plus the allowable uplift resistance provided by the soil. The resistance provided by the soil depends on the shear strength of the soil adjacent to the pier shaft and below the depth of the active zone. The allowable uplift resistance provided by the soils at the arch bridge site may be estimated using the allowable uplift capacity curves presented graphically in Appendix C (Figures C-1 through C-8). Side shear resistance was neglected to the elevation indicated in each of the notes of the uplift capacity curves.

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the sustained compressive load carried by that pier. We recommend that each pier be reinforced to withstand this net force or an amount equal to 0.5 percent of the cross-sectional area of the shaft, whichever is greater.

Lateral Resistance

Resistance to lateral loads and the expected pier behavior under the applied loading conditions will depend not only on subsurface conditions, but also on the loading conditions, the pier size(s), and the engineering properties of the pier. As this information is not yet available, analysis of pier behavior is not possible at this time. Once preliminary pier sizes, concrete strength, and reinforcement information are known, piers should be analyzed to determine the resulting lateral deflection, maximum bending moment, and ultimate bending moment. This type of analysis is typically performed utilizing a computer analysis program and usually requires a trial and error procedure to appropriately size the piers and meet project tolerances.

To assist the design engineer in this procedure, we are providing the soil parameters tabulated in the following table for use in analysis. These parameters are in accordance with the input requirements of one of the more commonly used computer programs for laterally-loaded piles, the "L-Pile Plus" program. If a different program is used for analysis, different parameters and different limitations may be required than what was assumed in selecting the parameters given on the following table. Thus, if a program other than "L-Pile Plus" is used, **RKCI** must be notified of the analysis method and the required soil parameters, so that we can review and revise our recommendations, if required.

The soil-related parameters required for input into the "L-Pile Plus" program are summarized in the following table:

Stratum (Soil Type)	**Approximate Depth Range (ft) ***	**c, tsf**	**** **(°)**	**$_{50}$**	**k_s, (pci)**	**k_c, (pci)**	**γ, (pcf)**
Poorly-Graded Sand Soils and Poorly-Graded Sand Soils with Silt	0 to 15	--	--	--	--	--	--
Poorly-Graded Sand Soils, Poorly-Graded Sand Soils with Silt, Silty Sand Soils, and Clayey Sand Soils (Below the Groundwater Table)	15 to 60	0	30	--	60	60	32.6
Lean Clay Soils, Sandy Fat Clay Soils, Fat Clay Soils with Sand, and Fat Clay Soils	60 to 80	2.0	0	0.005	1,000	400	32.6

* below the ground surface elevations existing at the time of our study.

Where:

c = undrained shear strength
 = angle of internal friction
$_{50}$ = strain at 50 percent
k_s = horizontal modulus of subgrade reaction (static)
k_c = horizontal modulus of subgrade reaction (cyclic)
γ = density (effective unit weight)

The values presented on the previous table for subgrade modulus and the strain at 50% are based on recommended values for the "L-Pile Plus" computer program for the strength of the subsurface conditions encountered in the borings, and are not necessarily based on laboratory test results.

The parameters presented in the previous table **do not** include factors of safety. Consequently, it is recommended that a factor of safety of at least 2 be introduced to the analysis by doubling the applied lateral loads and moments.

It should be noted that, in the event that piers are spaced closer than three shaft diameters center-to-center, a modification factor should be applied to the p-y curves to account for a group effect. We recommend the following p-Multipliers for the corresponding center-to-center pier spacings.

Spacing (in shaft diameters)	p-Multiplier
3	1.0
2	0.75
1	0.50

PIER SPACING

Where possible, we recommend that the piers be spaced at a center-to-center distance of at least three shaft diameters on-center for straight-shaft piers. Such spacing will not require a reduction in the load carrying capacity of the individual piers, as discussed in the previous section of this report.

If design and/or construction restraints require that piers be spaced closer than the recommended three shaft diameters, **RKCI** must re-evaluate the allowable bearing, skin friction, and lateral capacities presented previously for the individual piers. Reductions in load carrying capacities may be required depending upon individual loading and spacing conditions.

MAT FOUNDATIONS – LIFT STATION

It is our understanding, based on the information provided to us by others, that the proposed lift station structure will be located along the west side of Park Road 22, approximately 1,400 ft south of its intersection with Commodores Drive, and will be founded at a depth of about 20 ft below the ground surface elevation existing at the time of our study. As such, on the basis of the subsurface conditions encountered at the time of our field drilling activities, our field and laboratory testing, and our engineering analyses, the recommended maximum allowable soil-bearing pressure is 1,900 pounds per square foot (psf) for the lift station structure bearing at a depth of 20 ft below the ground surface elevation existing at the time of our study.

The maximum allowable soil-bearing pressure presented previously will provide a factor of safety of 3 with respect to the measured soil shear strength, provided that the subgrade is prepared in accordance with the recommendations outlined in the *Site Preparation* subsection of the *Foundation Construction Considerations* of this report.

Uplift forces due to groundwater can affect the performance of the lift station structure. The general contractor should evaluate the groundwater depth and pressure prior to starting construction. Consideration should be given to high groundwater levels immediately post construction.

Frictional forces between the concrete walls and the supporting soils should be neglected for the upper 10 feet. As such, the only resistance to potential uplift forces in this zone will be the self-weight of the concrete matrix. An allowable friction value of 250 psf may be used below a depth of 10 feet along the walls of the lift station, assuming competent surrounding soils and proper contact with these soils. The concrete thickness may be increased to increase dead load and/or the slab can be extended at the base to provide a lip or other measures considered to provide the required uplift resistance. We recommend using a buoyant unit weight of 90 pcf and 35 pcf for the concrete and soil, respectively, to compute uplift resistance.

Considering the depth of the proposed lift station, the subsurface conditions observed in the borings, and the depth to water, we recommend the caisson sinking method for constructing the lift station. In general, this method consists of three steps. 1) The caisson is constructed on the unexcavated site (often in stages), with the caisson comprising only the walls of the final structure. 2) Excavation inside the walls then proceeds, allowing the caisson to sink under its own weight. 3) Once the design depth is reached, the bottom slab is poured. Any steel struts needed to support the side walls can be installed at any time. However, supports should not impede the excavation process.

The caisson procedure eliminates the need for a temporary retention system or large, open cut excavations. Caissons can, however, experience problems with alignment and termination at the proper design depth. Due to the sandy soils encountered at this site and the depth of the water table, the stability of the bottom of the proposed excavation will be a concern and should be monitored during construction. The caisson sinking method may be completed in-the-dry or by wet slurry methods.

If dry a method is selected for construction, then dewatering will be required. An appropriate dewatering system as previously-discussed should be implemented outside the perimeter of the caisson area prior to sinking. The dewatering system should maintain the groundwater level at least 5 feet below the level of excavation throughout the period of excavation and construction of the structural slab. Of primary concern is the loss of fines from the stratum to the dewatering system. To reduce the loss of fines, an appropriate filtering system should be incorporated in the design of the well screens of the dewatering system.

If it is decided to construct the caisson by the wet method, then the differential hydrostatic pressure from the groundwater level within the retained soils is balanced by maintaining a sufficient head of water or slurry within the interior of the caisson during excavation. At all times during construction by the wet method, the level of water or slurry within the caisson should be maintained above the external water level.

In either case, the following guidelines should be considered by the design team:

- The caisson should be designed as a friction unit using an allowable side shear resistance value of 250 pounds per square feet for the portion of the shaft extending below a depth of 10 feet.

- The caisson should be designed to withstand the anticipated lateral earth pressures and a full hydrostatic profile (see the *Lateral Earth Pressure* Section of this report for appropriate earth pressure coefficients). This case refers to the working design when the lift station may be empty and the surrounding soils may be in a wet state.

- If the friction between the outside wall and the soil is high during installation, it may be necessary to assist the sinking by using jetting or adding weight to the top of caisson.

- Care should be taken to excavate the soil uniformly inside the caisson. The saturated sands observed in the boring performed for this study may run, or ravel, during excavation resulting in uneven sinking or tilting of the caisson.

- The use of a steel shoe with an angled edge for "cutting" into the soil should be considered.

- Protrusions from the caisson wall should be minimized. Dowels for the base slab should either be bent into a keyway (for small bars), or a threaded coupler should be used.

LATERAL EARTH PRESSURES

Equivalent fluid density values for computation of lateral soil pressures acting on below-grade structures such as the proposed sheetpiling were evaluated for various types of backfill materials that may be placed behind the below-grade structures. These values, as well as corresponding lateral earth pressure coefficients and estimated unit weights, are presented in the following table in preferential order for use as backfill materials.

Backfill Type	**Estimated Total Unit Weight (pcf)**	**Active Condition**		**At-Rest Condition**	
		Earth Pressure Coefficient, k_a	**Equivalent Fluid Density (pcf)**	**Earth Pressure Coefficient, k_o**	**Equivalent Fluid Density (pcf)**
Washed Gravel	135	0.29	40	0.45	60
Crushed Limestone	145	0.24	35	0.38	55
Clean Sand	120	0.33	40	0.50	60
Pit Run Clayey Gravels or On-Site Sands	135	0.32	45	0.48	65
Clays	110	0.59	65	0.74	80

The values tabulated in the previous table under "Active Condition" pertain to flexible sheetpiling free to tilt outward as a result of lateral earth pressures. For rigid, non-yielding sheetpiling the values under "At-Rest Condition" should be used.

The values presented in the previous table assume the surface of the backfill materials to be level. Sloping the surface of the backfill materials will increase the surcharge load acting on the below-grade structures. The values presented in the previous table also do not include the effect of surcharge loads such as construction equipment, vehicular loads, or future storage near the below-grade structures. Nor do the values account for hydrostatic pressures resulting from the shallow groundwater at this site or from groundwater seepage entering and ponding within the backfill materials. However, these surcharge loads and groundwater pressures should be considered in designing any below-grade structures subjected to lateral earth pressures.

The use of expansive soils as backfill against the proposed below-grade structures is not recommended. These soils generally provide higher design active earthen pressures, as indicated previously, but may also exert additional active pressures associated with swelling. Controlling the moisture and density of these materials during placement will help reduce the likelihood and magnitude of future active pressures due to swelling, but this is no guarantee.

BACKFILL COMPACTION

Placement and compaction of backfill behind the below-grade structures will be critical, particularly at locations where deep backfill will support adjacent near-grade foundations and/or flatwork. If the backfill is not properly compacted in these areas, the adjacent foundations/flatwork can be subject to settlement.

To reduce potential settlement of adjacent foundations/flatwork, the backfill materials should be placed and compacted as recommended in the *Select Fill* subsection of the *Construction Considerations* section of this report. Each lift or layer of the backfill should be tested during the backfilling operations to document the degree of compaction. Within at least a 5-ft zone of the structures, we recommend that compaction be accomplished using hand-guided compaction equipment capable of achieving the maximum dry density in a series of 3 to 5 passes.

WATERPROOFING

Consideration may be given to applying waterproofing coatings to any below-grade walls. Waterproofing of the below-grade walls for capillary moisture is often accomplished by painting the wall exteriors with a bituminous material. For greater seepage protection, membrane waterproofing would be required.

FOUNDATION CONSTRUCTION CONSIDERATIONS

SITE DRAINAGE

Drainage is an important key to the successful performance of any foundation. Good surface drainage should be established prior to and maintained after construction to help prevent water from ponding within or adjacent to the proposed structure foundations, and to facilitate rapid drainage away from the proposed structure foundations. Failure to provide positive drainage away from the structures can result in localized differential vertical movements in the soil-supported foundations and floor slabs.

Other drainage and subsurface drainage issues are discussed in the *Foundation Analyses* section of this report.

DEWATERING

The structures associated with this project will require excavations extending down to elevations ranging from about 6.0 ft to 16.0 ft below MSL. Further, groundwater was encountered during our drilling operations at elevations ranging from about 0.8 ft below MSL to 5.0 ft above MSL.

Based on the subsurface conditions encountered in the borings, and the typical fluctuations in groundwater levels in this region, groundwater will be encountered during construction activities. Dewatering methods, such as deep wells and/or well point systems as well as temporary earthen berms and conventional sump-and-pump dewatering methods will be required for the control of groundwater seepage. The dewatering system should maintain the groundwater level at least 5 feet below the level of excavation throughout the period of excavation and construction. Excavations below the groundwater table, generally require lowering the piezometric level to permit construction in a relatively dry state. This should be performed to control seepage into the excavations and to reduce artesian water pressures below the bottom of the excavations. Wellpoints, deep wells, and/or submersible pump and sump dewatering systems are commonly used. The implementation of these methods should be anticipated for construction of these structures. The design of dewatering systems is beyond the scope of this study. Proper construction procedures and equipment will be critical for proper performance of the dewatered foundation excavations. Additionally, protection of personnel entering the excavations and providing a dry, stable subgrade on which to construct foundations will be crucial.

SITE PREPARATION

The proposed structure areas and all areas to support select fill should be stripped of all vegetation and/or organic topsoil down to a minimum depth of 8 inches and extending a minimum of 5 ft beyond the structure footprint areas.

Exposed subgrades should be thoroughly proofrolled in order to locate and densify any weak, compressible zones. A minimum of 5 passes of a fully-loaded dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or his/her representative to document subgrade conditions and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with a suitable, compacted select fill in accordance with the recommendations presented under the *Select Fill* subsection of this section of the report. Proofrolling operations and any excavation/backfill activities should be observed by **RKCI** representatives to document subgrade condition and preparation.

Upon completion of the proofrolling operations and just prior to fill placement, the exposed subgrades should be moisture-conditioned by scarifying to a minimum depth of 8 in. and recompacting to a minimum of 95 percent of the maximum dry density as determined from the ASTM D1557, Compaction Test. The moisture content of the subgrade should be maintained within the range of two percentage points below the optimum moisture content to two percentage points above the optimum moisture content until permanently covered.

EROSION PROTECTION

We understand that the excavated materials from the proposed water exchange channel areas are planned to be used as fill material to raise the reconstructed road alignment above the proposed arch bridge structures. Properly-compacted sands are generally resistant to erosion, although all soils will erode if subjected to a sufficiently swift current, tidal flow, overtopping, or unusual turbulence. An allowance of about 2 ft is recommended in order to accommodate for excessive scour or over dredging. We also recommend that the proposed sheetpiling be installed deeper than its conceptual elevation of 16 ft, at the interface of the water exchange channel and the sidewalk at each end of the arch structures. In general, for this type of cantilever sheetpile design, the sheetpiling depth is typically twice as deep as the exposed height of the sheetpile structure.

SELECT FILL

If utilized, materials used as select fill for final site grading preferably should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2004 Standard Specification for Construction and Maintenance of Highways, Streets, and Bridges, Item 247, Flexible Base, Type A, Type B, or Type C, Grades 1 through 3.

Alternatively, the following soils, as classified according to the USCS, may be considered satisfactory for use as select fill materials at this site: SC, GC, and combinations of these soils. In addition to the USCS classification, alternative select fill materials shall have a maximum liquid limit of 35 percent, a plasticity index between 5 and 17 percent, and a maximum particle size not exceeding 4 inches or one-half the loose lift thickness, whichever is smaller. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a minimum rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with pit-run materials.

If the previously-listed alternative materials are being considered for bidding purposes, the materials should be submitted to the Geotechnical Engineer for pre-approval a minimum of 10 working days or more prior to the bid date. Failure to do so will be the responsibility of the General Contractor. The General Contractor will also be responsible for ensuring that the properties of all delivered alternate select fill materials are similar to those of the pre-approved submittal. It should also be noted that when using alternative fill materials, difficulties may be experienced with respect to moisture control during and subsequent to fill placement, as well as with erosion, particularly when exposed to inclement weather. This may result in sloughing of beam trenches and/or pumping of the fill materials.

Soils classified as CH, CL, MH, ML, SM, GM, OH, OL, and Pt under the USCS and not meeting the alternative select fill material requirements, are **not** considered suitable for use as select fill materials at this site.

Select fill should be placed in loose lifts **not** exceeding 8 in. in thickness and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. The moisture content of the fill should be maintained within the range of two percentage points below the optimum moisture content to two percentage points above the optimum moisture content until the final lift of fill is permanently covered.

The select fill should be properly compacted in accordance with these recommendations and tested by **RKCI** personnel for compaction as specified.

SHALLOW EXCAVATIONS

Shallow excavations should be observed by the Geotechnical Engineer or his/her representative prior to placement of reinforcing steel and concrete. This is necessary to document that the bearing soils at the bottom of the excavations are similar to those encountered in the borings and that excessive loose materials and water are not present in the excavations. If loose soil pockets are encountered in the foundation excavations, they should be removed and replaced with a compacted non-expansive fill material or lean concrete up to the design foundation bearing elevation.

DRILLED PIERS

Drilled pier excavations must be examined by an **RKCI** representative who is familiar with the geotechnical aspects of the subsurface stratigraphy, the structural configuration, foundation design details, and assumptions prior to placing concrete. This is to observe that:

- The shaft has been excavated to the specified dimensions at the correct depth established by the previously mentioned criteria;
- The shaft has been drilled plumb within specified tolerances along its total length; and
- Excessive cuttings, buildup and soft, compressible materials have been removed from the bottom of the excavation.

Drilled pier excavation observations should be scheduled with the Geotechnical Engineer a minimum of 48 hours prior to pier drilling. Failure to do so will be the responsibility of the General Contractor.

Due to the presence of very dense sands below a depth of about 45 ft below the ground surface elevation existing at the time of our study, high-powered, high-torque drilling equipment should be anticipated for drilled pier construction at this site.

Reinforcement and Concrete Placement

Reinforcing steel should be checked for size and placement prior to concrete placement. Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the moisture content or the state of stress of the foundation materials. Concrete should not be placed in the pier excavations without the approval of the Engineer. No foundation element should be left open overnight without concreting.

Temporary Casing

Groundwater was observed in the borings at the time of our drilling operations at depths ranging from about 1.5 ft to 3 ft below the ground surface elevations at the time of our study. Thus, groundwater seepage and/or side sloughing will be encountered at the time of construction, depending on climatic conditions prevalent at the time of construction. Therefore, we recommend that the bid documents require the foundation contractor to specify unit costs for different lengths of casing and/or slurry drilling techniques which will be required.

EXCAVATION SLOPING AND BENCHING

Excavations that extend to or below a depth of 5 ft below construction grade shall require the General Contractor to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, are beyond the scope of the current study. Any such designs and safety plans shall be developed in accordance with current Occupational Safety and Health Administration (OSHA) guidelines and other applicable industry standards.

To assist in preparing an excavation safety plan, we have classified the soils encountered at these sites based on the data collected during this study. The soils encountered at these sites are classified as Type "C" soils under current OSHA regulations pertaining to excavations. In excavations penetrating these soils, the sloping and benching schemes specified for Type "C" soils under the OSHA regulations require that the excavation sidewalls be sloped no steeper than 1-1/2:1 (horizontal:vertical).

EXCAVATION EQUIPMENT

Very dense sands will likely be encountered during deep foundation construction at this site, thus the need for high-powered, high-torque drilling equipment should be anticipated for this project. The boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that General Contractors and their subcontractors interested in bidding on the work perform their own tests in the form of test piers and/or test pits to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

UTILITIES

Utilities which project through slab-on-grade, slab-on-fill, "floating" floor slabs, or any other rigid unit should be designed with either some degree of flexibility or with sleeves. Such design features will help reduce the risk of damage to the utility lines.

Our experience indicates that significant settlement of backfill can occur in utility trenches, particularly when trenches are deep, when backfill materials are placed in thick lifts with insufficient compaction, and when water can access and infiltrate the trench backfill materials. The potential for water to access the backfill is increased where water can infiltrate flexible base materials due to insufficient penetration of curbs, and at sites where geological features can influence water migration into utility trenches. It is our belief that another factor which can significantly impact settlement is the migration of fines within the backfill into the open voids in the underlying free-draining bedding material.

To reduce the potential for settlement in utility trenches, we recommend that consideration be given to the following:

- Backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and backfilling procedures should be tested and documented.
- Curbs should be installed to a sufficient depth to reduce water infiltration beneath the curbs into the pavement flexible base materials (see also the *Foundation Analyses* section of this report).
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N or CONTECH C-Drain Geocomposite) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.

PAVEMENT RECOMMENDATIONS

Recommendations for flexible pavements for a 20-year design period are presented in this report. Drainage conditions will have a significant impact on long-term performance, particularly where permeable flexible base materials are utilized in the pavement section. Drainage considerations are discussed in more detail in a subsequent section of this report.

SUBGRADE CONDITIONS

A single generalized subgrade condition has been assumed for this site. The predominant subgrade soils used in developing the pavement sections for this project are the surficial sand soils. A laboratory CBR value of 19 was measured for these subgrade soils under inundated conditions. On the basis of our past experience with similar subsurface conditions in this area, a design CBR value of 10 was assigned to evaluate the flexible pavement components. This design CBR value assumes that the subgrade soils will be prepared in accordance with the recommendations stated in the *Subgrade Preparation* subsection of the *Pavement Construction Guidelines* section of this report.

DESIGN INFORMATION

The following recommendations for the pavement sections are based on our past experience with similar subgrade soils; traffic loading provided to us by TxDOT; a CBR value based on laboratory testing for the subgrade soils; and design procedures utilizing a software program entitled “FPS 19W, Flexible Pavement Design System”, published by TxDOT.

As previously stated, based on the traffic information provided to us via electronic-mail transmittal on Wednesday, March 14, 2012 from TxDOT, we understand that an ADT of 17,054 was estimated for 2011, and an ADT of 24,201 is projected for 2031, with 19.7% truck ADT, which equates to about 6,412,000 ESAL’s for a 20-year design period.

FLEXIBLE PAVEMENTS

The following equivalent flexible pavement sections are available for this site:

Flexible Pavement Alternatives	Prepared Subgrade (in.)	FBM (in.)	HMAC (in.)
I	12	17	2-1/2
II	12	16-1/2	3
III	12	15-1/2	3-1/2
IV	12	14-1/2	4

Where:
PS = Prepared Subgrade
FBM = Flexible Base Material
HMAC = Hot-Mix Asphaltic Concrete Surface Course

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

Areas to support pavements should be stripped of all vegetation and/or organic topsoil down to a minimum depth of 8 inches and extend a minimum of 2 ft beyond the pavement perimeters. Upon completion of site stripping activities, the exposed subgrade should be thoroughly proofrolled in accordance with the *Site Preparation* subsection recommendations provided in the *Foundation Construction Considerations* section of this report. Likewise, upon completion of the proofrolling activities and just prior to select fill or flexible base placement, the exposed subgrade should be scarified and recompacted as recommended in such subsection.

DRAINAGE CONSIDERATIONS

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

1) Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.

2) Final site grading should eliminate isolated depressions adjacent to curbs which may allow surface water to pond and infiltrate into the underlying soils. **Curbs should completely penetrate flexible base materials and should be installed to sufficient depth to reduce infiltration of water beneath the curbs**.

3) Pavement surfaces should be maintained to help minimize surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

ON-SITE SAND FILL

The pavement recommendations presented in this report were prepared assuming that on-site soils will be used for site grading in the proposed pavement areas. If used, we recommend that on-site soils be placed in loose lifts not exceeding 8 in. in thickness and compacted to a minimum of 95 percent of the maximum dry density as determined from ASTM D1557. The moisture content of the subgrade should be maintained within the range of two percentage points below the optimum moisture content to two percentage points above the optimum moisture content until permanently covered. We recommend that on-site soil fill materials be free of roots and other organic or degradable material. We also recommend that the maximum particle size not exceed 4 in. or one half the lift thickness, whichever is smaller.

SELECT FILL

If implemented, select fill materials utilized for achieving finished subgrade elevations in pavement areas should be in accordance with the *Select Fill* subsection recommendations provided in the *Foundation Construction Considerations* section of this report.

FLEXIBLE BASE COURSE

The flexible base course should consist of material conforming to TxDOT 2004 Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, Item 247, Flexible Base, Types A, B, or C, Grades 1 through 3.

The flexible base course should be placed in lifts with a maximum compacted thickness of 8 in. and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557. The moisture content of the base course materials should be maintained within the range of three percentage points below the optimum moisture content to three percentage points above the optimum moisture content until permanently covered.

ASPHALTIC CONCRETE SURFACE COURSE

The asphaltic concrete surface course should conform to TxDOT 2004 Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, Item 340, Dense-Graded Hot-Mix Asphalt (Method), Type D. The asphaltic concrete should be compacted to a minimum of 92 percent of the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS ENGINEERING AND TESTING SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, **Raba Kistner**, is retained to perform the construction materials engineering and testing services during the construction of the project. This is because:

- **RKCI** has an intimate understanding of the geotechnical engineering report's findings and recommendations. **RKCI** understands how the report should be interpreted and can provide such interpretations on site, on the CLIENT's behalf.
- **RKCI** knows what subsurface conditions are anticipated at the site.
- **RKCI** is familiar with the goals of the CLIENT and the project's design professionals, having worked with them in the development of the project's geotechnical design workscope. This enables **RKCI** to suggest remedial measures (when needed) which help meet others' requirements.
- **RKCI** has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors' work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.
- **RKCI** cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction materials engineering and testing services. At the appropriate time before construction, we advise that **RKCI** and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected General Contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the General Contractor. **RKCI** looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.

* * * * * * * * * * * * * * * * * *

The following figures are attached and complete this report:

APPENDIX A

Figure A-1	Boring Location Map
Figures A-2 through A-11	Drilling Logs
Figures A-12 and A-13	Grain Size Distributions
Figure A-14	Moisture-Density Relationship
Figures A-15 and A-16	Consolidation Test Reports
Figures A-17 through A-20	Multi Stage Triaxial Undrained Compression Test Results

APPENDIX B

Figure B-1	Boring Location Map
Figures B-2 through B-8	Log of Borings
Figures B-9	Key to Terms and Symbols
Figures B-10	Results of Soil Sample Analyses

APPENDIX C

Figure C-1 through C-8	Drilled Pier Capacity Curves

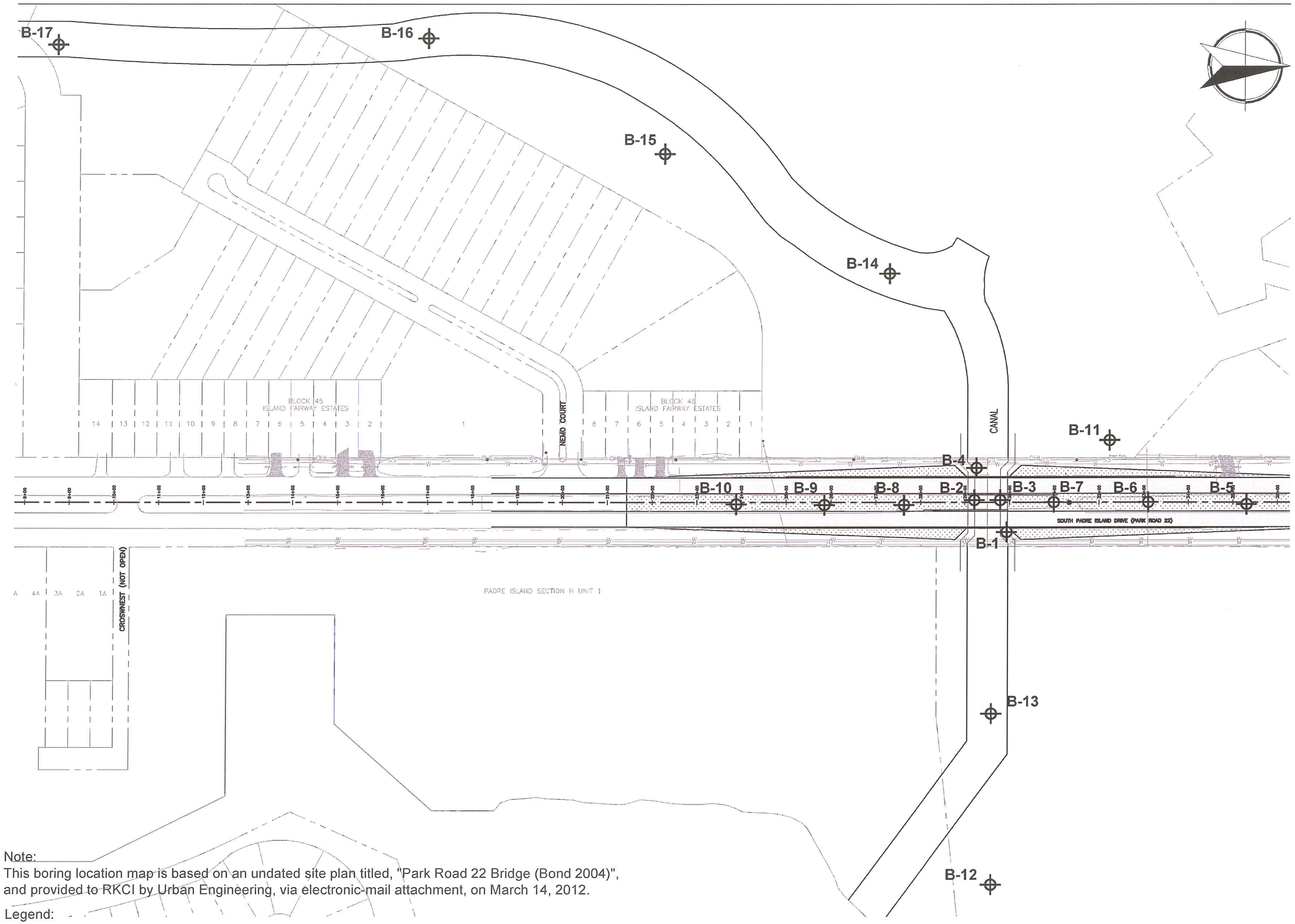
Important Information About Your Geotechnical Engineering Report

ATTACHMENTS

APPENDIX A

Raba Kistner
Engineering • Testing • Environmental
Facilities • Infrastructure
800 E. Hackberry Avenue
McAllen, Texas 78501
(956)682-5332 TEL
(956)682-5487 FAX
www.rkci.com
TBPE Firm F-3257

BORING LOCATION MAP
PARK ROAD 22 BRIDGE PROJECT
ALONG SOUTH PADRE ISLAND DRIVE
CORPUS CHRISTI, NUECES COUNTY, TEXAS



Note:
This boring location map is based on an undated site plan titled, "Park Road 22 Bridge (Bond 2004)", and provided to RKCI by Urban Engineering, via electronic-mail attachment, on March 14, 2012.

Legend:
B-1 through B-4 and B-12 through B-17 boring logs are included in Appendix A
B-5 through B-11 boring logs are included in Appendix B

REVISIONS:

No.	DATE	DESCRIPTION

PROJECT No.: ACA11-007-00

ISSUE DATE:	03-14-12
DRAWN BY:	NES
CHECKED BY:	JLP
REVIEWED BY:	KML

FIGURE
A-1

O:\Active Projects\McAllen\2011\ACA - Corpus Christi\ACA11-007-00 Park Road 22\Drawings\Figure 1 - 11x17.dwg

DRILLING LOG

WinCore
Version 3.0

County	Nueces	Hole	B-1	District	Corpus Christi
Highway	358	Structure	Water Exchange Channel	Date	01/23/12
CSJ	Not Provided	Station	29+92.42	Grnd. Elev.	2.26 ft
		Offset	RT 67.66	GW Elev.	-0.24 ft

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test		Properties				Additional Remarks
				Lateral Press. (psi)	Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	
			SAND, poorly-graded, loose to compact, semi-moist to wet, dark grayish-brown, with seaweeds extending down to a depth of about 2 ft (SP)			22				
						27				PI = Non-Plastic
5		10 (6) 9 (6)								
						25				- becomes grayish-brown in color below a depth of about 7 ft -200% = 2
10		6 (6) 9 (6)								
						32				
15		14 (6) 14 (6)								
						23				-200% = 5
20		5 (6) 3 (6)								
						24				
25		23 (6) 19 (6)								
						24				PI = Non-Plastic
-25.7			SAND, silty, compact to loose, wet, gray (SM)							
30		23 (6) 23 (6)								
						22				-200% = 20
35		5 (6) 3 (6)								
						26	29	14	122	UNC = 0.29 tsf UNC = Unconfined Compressive Strength -200% = 20
-34.7			SAND, clayey, loose, wet, bluish-brown (SC)							
40		7 (6) 11 (6)								
-39.7			CLAY, sandy, lean, very stiff, wet, bluish-brown (CL)							
						26	30	15	121	-200% = 64
45		30 (6) 49 (6)								

Remarks:
The ground surface elevation was provided to us by Urban Engineering via electronic-mail attachment on Wednesday, March 14, 2012.
Any ground water elevation information provided on this boring log is representative of conditions existing on the day and for the specific location where this information was collected. The actual groundwater elevation may fluctuate due to time, climatic conditions, and/or construction activity.

Driller: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

	County	Nueces	Hole	B-1	District	Corpus Christi
WinCore	Highway	358	Structure	Water Exchange Channel	Date	01/23/12
Version 3.0	CSJ	Not Provided	Station	29+92.42	Grnd. Elev.	2.26 ft
			Offset	RT 67.66	GW Elev.	-0.24 ft

Elev. (ft)		LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test		Properties				Additional Remarks
					Lateral Press. (psi)	Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	
				CLAY, sandy, lean, very stiff, wet, bluish-brown (CL)							
-44.7				SAND, poorly-graded, with silt, dense to slightly compact, wet, bluish-brown (SP-SM)			26				
	50		50 (5) 50 (3)								
							26				-200% = 8
	55		19 (6) 18 (6)								
-54.7				CLAY, fat, stiff, wet, bluish-brown to grayish-brown, with sand (CH)			33			114	
	60		12 (6) 11 (6)								
							26	66	44	118	-200% = 87
-62.7	65		12 (6) 12 (6)	CLAY, sandy, fat, stiff to very hard, wet, bluish-brown to grayish-brown (CH)							
							39	63	46	117	-200% = 64
	70		13 (6) 37 (6)								
							29			116	UNC = 0.49 tsf
	75		47 (6) 50 (5)								
							21				where UNC = Unconfined Compressive Strength where UNC = Unconfined Compressive Strength
-77.7	80		50 (5) 50 (4)								
	85										
	90										

Remarks:

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Driler: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

WinCore
Version 3.0

County	Nueces	Hole	B-2	District	Corpus Christi
Highway	358	Structure	Bridge	Date	01/27/12
CSJ	Not Provided	Station	29+24.47	Grnd. Elev.	2.62 ft
		Offset	LT 2.17	GW Elev.	0.62 ft

Elev. (ft)		Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			SAND, poorly-graded, loose, dark brown, semi-moist, with seaweeds (SP)			40				
0.6			SAND, silty, loose to very loose, wet, light grayish-brown (SM)			27				PI = Non-Plastic
	5	6 (6) 7 (6)								
						30				-200% = 17
	10	3 (6) 3 (6)								
						30				
-12.4	15	16 (6) 20 (6)	SAND, poorly-graded, with silt, slightly compact to loose, wet, light grayish-brown (SP-SM)							
						25				-200% = 6
	20	7 (6) 6 (6)								
						28				
-22.4	25	15 (6) 21 (6)	SAND, silty, slightly compact to compact, wet, gray (SM)							
						21				-200% = 18
	30	27 (6) 24 (6)								
						23				PI = Non-Plastic
-32.4	35	5 (6) 4 (6)	SAND, poorly-graded, with silt, loose, wet, light grayish-brown (SP-SM)							
						25	23	10		-200% = 5
-37.4	40	16 (6) 20 (6)	SAND, clayey, slightly compact, wet, light grayish-brown (SC)							
						25	23	10		
	45	50 (6) 50 (4)								

Remarks:
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Driler: JEDI Pumps Logger: A. Rodriguez Organization: Raba Kistner Consultants, Inc.

DRILLING LOG

WinCore
Version 3.0

County	Nueces	Hole	B-2	District	Corpus Christi
Highway	358	Structure	Bridge	Date	01/27/12
CSJ	Not Provided	Station	29+24.47	Grnd. Elev.	2.62 ft
		Offset	LT 2.17	GW Elev.	0.62 ft

Elev. (ft)		LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test		Properties				Additional Remarks
					Lateral Press. (psi)	Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	
-42.4				SAND, poorly-graded, dense, wet, grayish-brown (SP)			27	29	16		-200% = 5
-47.4	50		32 (6) 24 (6)	SAND, clayey, compact to slightly compact, wet, grayish-brown to light grayish-brown, with shell fragments (SC)			18				
	55		16 (6) 16 (6)				16				- with gypsum nodules below a depth of about 55 ft -200% = 47
-57.4	60		12 (6) 14 (6)								
	65										
	70										
	75										
	80										
	85										
	90										

Remarks:

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Driller: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

WinCore Version 3.0	County	Nueces	Hole	B-3	District	Corpus Christi
	Highway	358	Structure	Bridge	Date	01/26/12
	CSJ	Not Provided	Station	29+78.59	Grnd. Elev.	2.49 ft
			Offset	LT 2.23	GW Elev.	0.49 ft

Elev. (ft)		Texas Cone Penetrometer	Strata Description	Triaxial Test		Properties				Additional Remarks
				Lateral Press. (psi)	Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	
			SAND, poorly-graded, loose, wet, dark grayish-brown, with seaweeds (SP)			23				-200% = 3
						28				
0.5			SAND, poorly-graded, loose to very loose to slightly compact to loose, wet, light grayish-brown (SP)							
	5	6 (6) 8 (6)								
						49				PI = Non-Plastic
	10	3 (6) 2 (6)								
						28				
	15	15 (6) 21 (6)								
						27				-200% = 5
	20	6 (6) 6 (6)								
						29				
-22.5	25	16 (6) 20 (6)	SAND, clayey, slightly compact, wet, grayish-brown (SC)							
						20	27	7		
-27.5	30	26 (6) 25 (6)	SAND, poorly-graded, with silt, compact to loose to dense, wet, grayish-brown (SP-SM)							
						24				
	35	4 (6) 4 (6)								
						25				-200% = 8
	40	15 (6) 21 (6)								
						26				-200% = 16
	45	49 (6) 50 (3)								

Remarks:

The ground surface elevation was provided to us by Urban Engineering via electronic-mail attachment on Wednesday, March 14, 2012.

Any ground water elevation information provided on this boring log is representative of conditions existing on the day and for the specific location where this information was collected. The actual groundwater elevation may fluctuate due to time, climatic conditions, and/or construction activity.

Driler: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

WinCore
Version 3.0

County	Nueces	Hole	B-3	District	Corpus Christi
Highway	358	Structure	Bridge	Date	01/26/12
CSJ	Not Provided	Station	29+78.59	Grnd. Elev.	2.49 ft
		Offset	LT 2.23	GW Elev.	0.49 ft

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			SAND, poorly-graded, with silt, compact to loose to dense, wet, grayish-brown (SP-SM)			25				-200% = 9
-47.5 50		34 (6) 25 (6)								
			SAND, clayey, compact to slightly compact, wet, bluish-brown, with gypsum nodules (SC)			19	36	24		
55		15 (6) 16 (6)								
						23				-200% = 47
-57.5 60		12 (6) 15 (6)								
			CLAY, fat, stiff to hard, light grayish-brown, with orange ferrous stains (CH)			37	80	50		
65		15 (6) 14 (6)								
						35				
70		19 (6) 23 (6)								
						35				-200% = 87
75		30 (6) 42 (6)								
						39	102	77		
-77.5 80		33 (6) 50 (5)								
85										
90										

Remarks:
The ground surface elevation was provided to us by Urban Engineering via electronic-mail attachment on Wednesday, March 14, 2012.
Any ground water elevation information provided on this boring log is representative of conditions existing on the day and for the specific location where this information was collected. The actual groundwater elevation may fluctuate due to time, climatic conditions, and/or construction activity.

Driler: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

WinCore Version 3.0					
County	Nueces	Hole	B-4	District	Corpus Christi
Highway	358	Structure	Bridge	Date	01/26/12
CSJ	Not Provided	Station	29+26.33	Grnd. Elev.	2.53 ft
		Offset	LT 76.83	GW Elev.	0.03 ft

Elev. (ft)		LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
				SAND, poorly-graded, very loose, semi-moist, dark grayish-brown, with seaweeds (SP)			24				
0.5				SAND, silty, very loose to loose, wet, light grayish-brown (SM)			18				PI = Non-Plastic
	5		2 (6) 1 (6)								
							28				-200% = 13
	10		8 (6) 10 (6)								
							27				
-12.5	15		12 (6) 19 (6)	SAND, poorly-graded, slightly compact to loose to slightly compact, wet, light grayish-brown to dark grayish-brown (SP)							
							23				PI = Non-Plastic
	20		8 (6) 6 (6)								
							26				-200% = 1
	25		16 (6) 16 (6)								
							26				
-27.5	30		40 (6) 32 (6)	SAND, clayey, compact to loose, wet, bluish-brown (SC)							
							20				-200% = 21
	35		5 (6) 4 (6)								
							25	38	24	120	-200% = 27
-37.5	40		9 (6) 10 (6)	SAND, poorly-graded, with silt, loose to compact to slightly compact, wet, dark grayish-brown (SP-SM)							
							21				
	45		32 (6) 39 (6)								

Remarks:
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Driller: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

	County	Nueces	Hole	B-4	District	Corpus Christi
WinCore	Highway	358	Structure	Bridge	Date	01/26/12
Version 3.0	CSJ	Not Provided	Station	29+26.33	Grnd. Elev.	2.53 ft
			Offset	LT 76.83	GW Elev.	0.03 ft

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test		Properties				Additional Remarks
				Lateral Press. (psi)	Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	
			SAND, poorly-graded, with silt, loose to compact to slightly compact, wet, dark grayish-brown (SP-SM)			22				
50		31 (6) 26 (6)								
						22				-200% = 12
-52.5 55		14 (6) 17 (6)	CLAY, lean, stiff, wet, grayish-brown (CL)							
						26	31	18		
-57.5 60		13 (6) 14 (6)								
65										
70										
75										
80										
85										
90										

Remarks:

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Driler: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

WinCore	County	Nueces	Hole	B-12	District
Version 3.0	Highway	358	Structure	Water Exchange Channel	Date
	CSJ	Not Provided	Station	Not Provided	Grnd. Elev.
			Offset	Not Provided	GW Elev.

District	Corpus Christi
Date	01/30/12
Grnd. Elev.	3.48 ft
GW Elev.	1.98 ft

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test		Properties				Additional Remarks
				Lateral Press. (psi)	Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	
			SAND, poorly-graded, loose, semi-moist to wet, light brown to brown (SP)			20				PI = Non-Plastic
						23				-200% = 3
-0.5		3 (6) 5 (6)	SAND, silty, loose to compact, wet, brown to grayish-brown (SM)							
5										
						26				
10		6 (6) 4 (6)								
						28				PI = Non-Plastic
15		16 (6) 20 (6)								
						24				
20		5 (6) 5 (6)								
						22				-200% = 13
25		16 (6) 20 (6)								
						24				
-26.5 30		16 (6) 26 (6)								
35										
40										
45										

Remarks:

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Driler: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

WinCore Version 3.0					
County	Nueces	Hole	B-13	District	Corpus Christi
Highway	358	Structure	Water Exchange Channel	Date	01/31/12
CSJ	Not Provided	Station	Not Provided	Grnd. Elev.	2.40 ft
		Offset	Not Provided	GW Elev.	0.90 ft

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test		Properties				Additional Remarks
				Lateral Press. (psi)	Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	
			SAND, poorly-graded, loose, semi-moist, light brown (SP)			20				-200% = 3
0.4			SAND, silty, loose to compact, wet, dark brown to grayish-brown, with seaweed extending down to a depth of about 5 ft (SM)			26				PI = Non-Plastic
5		6 (6) 7 (6)								
						31				
10		8 (6) 11 (6)								
						26				
15		23 (6) 28 (6)								
						25				PI = Non-Plastic
20		7 (6) 11 (6)								
						24				
25		19 (6) 25 (6)								
						19				-200% = 16
-27.6 30		23 (6) 19 (6)								
35										
40										
45										

Remarks:

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Any ground water elevation information provided on this boring log is representative of conditions existing on the day and for the specific location where this information was collected. The actual groundwater elevation may fluctuate due to time, climatic conditions, and/or construction activity.

Driler: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

1 of 1

	County	Nueces	Hole	B-14	District	Corpus Christi
WinCore	Highway	358	Structure	Water Exchange Channel	Date	01/31/12
Version 3.0	CSJ	Not Provided	Station	Not Provided	Grnd. Elev.	4.89 ft
			Offset	Not Provided	GW Elev.	2.39 ft

Elev. (ft)		LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test		Properties				Additional Remarks
					Lateral Press. (psi)	Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	
				SAND, poorly-graded, loose, semi-moist to wet, brown to dark brown, with seaweeds extending down to a depth of about 2 ft (SP)			12 11				
	5		6 (6) 4 (6)								
							26				-200% = 1
-5.1	10		7 (6) 13 (6)	SAND, poorly-graded, with silt, loose to slightly compact to loose, wet, grayish-brown (SP-SM)							
							25				PI = Non-Plastic
	15		7 (6) 4 (6)								
							24				-200% = 8
	20		12 (6) 12 (6)								
							27				PI = Non-Plastic
	25		8 (6) 8 (6)								
							22				- with large amount of gypsum nodules below a depth of 27 ft
-25.1	30		6 (6) 9 (6)								
	35										
	40										
	45										

Remarks:

The ground surface elevation was provided to us by Urban Engineering via electronic-mail attachment on Wednesday, March 14, 2012.

Any ground water elevation information provided on this boring log is representative of conditions existing on the day and for the specific location where this information was collected. The actual groundwater elevation may fluctuate due to time, climatic conditions, and/or construction activity.

Driller: JEDI Pumps Logger: A. Rodriguez Organization: Raba Kistner Consultants, Inc.

DRILLING LOG

		County	Nueces	Hole	B-15	District	Corpus Christi
WinCore		Highway	358	Structure	Water Exchange Channel	Date	01/31/12
Version 3.0		CSJ	Not Provided	Station	Not Provided	Grnd. Elev.	3.86 ft
				Offset	Not Provided	GW Elev.	1.36 ft

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			SAND, poorly-graded, loose, semi-moist, brown, with seaweeds (SP)			14 11				PI = Non-Plastic -200% = 1
1.9			SAND, poorly-graded, loose to slightly compact, wet, brown to grayish-brown (SP)							
5		6 (6) 7 (6)				26				
10		8 (6) 11 (6)				26				
15		3 (6) 5 (6)				28				
20		6 (6) 5 (6)				26				-200% = 4
25		16 (6) 17 (6)				22				- with large amount of gypsum nodules and clay lenses below a depth of about 27 ft PI = Non-Plastic
-26.1 30		15 (6) 17 (6)								
35										
40										
45										

Remarks:

The ground surface elevation was provided to us by Urban Engineering via electronic-mail attachment on Wednesday, March 14, 2012.

Any ground water elevation information provided on this boring log is representative of conditions existing on the day and for the specific location where this information was collected. The actual groundwater elevation may fluctuate due to time, climatic conditions, and/or construction activity.

Driler: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

WinCore
Version 3.0

County	Nueces	Hole	B-16	District	Corpus Christi
Highway	358	Structure	Water Exchange Channel	Date	02/1/12
CSJ	Not Provided	Station	Not Provided	Grnd. Elev.	4.37 ft
		Offset	Not Provided	GW Elev.	2.37 ft

Elev. (ft)	LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
			SAND, poorly-graded, loose, semi moist, dark brown, with seaweeds (SP)			24				-200% = 2
						28				
2.4			SAND, poorly-graded, loose to dense to compact, wet, brown to grayish-brown (SP)							
5		4 (6) 5 (6)								
						27				PI = Non-Plastic
10		8 (6) 10 (6)								
						28				-200% = 3
15		5 (6) 5 (6)								
						28				
20		8 (6) 10 (6)								
						25				
25		50 (3) 50 (3)								
						24				PI = Non-Plastic
-25.6 30		27 (6) 24 (6)								
35										
40										
45										

Remarks:

The ground surface elevation was provided to us by Urban Engineering via electronic-mail attachment on Wednesday, March 14, 2012.

Any ground water elevation information provided on this boring log is representative of conditions existing on the day and for the specific location where this information was collected. The actual groundwater elevation may fluctuate due to time, climatic conditions, and/or construction activity.

Driler: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**

DRILLING LOG

	County	Nueces	Hole	B-17	District	Corpus Christi
WinCore	Highway	358	Structure	Water Exchange Channel	Date	02/1/12
Version 3.0	CSJ	Not Provided	Station	Not Provided	Grnd. Elev.	6.99 ft
			Offset	Not Provided	GW Elev.	4.99 ft

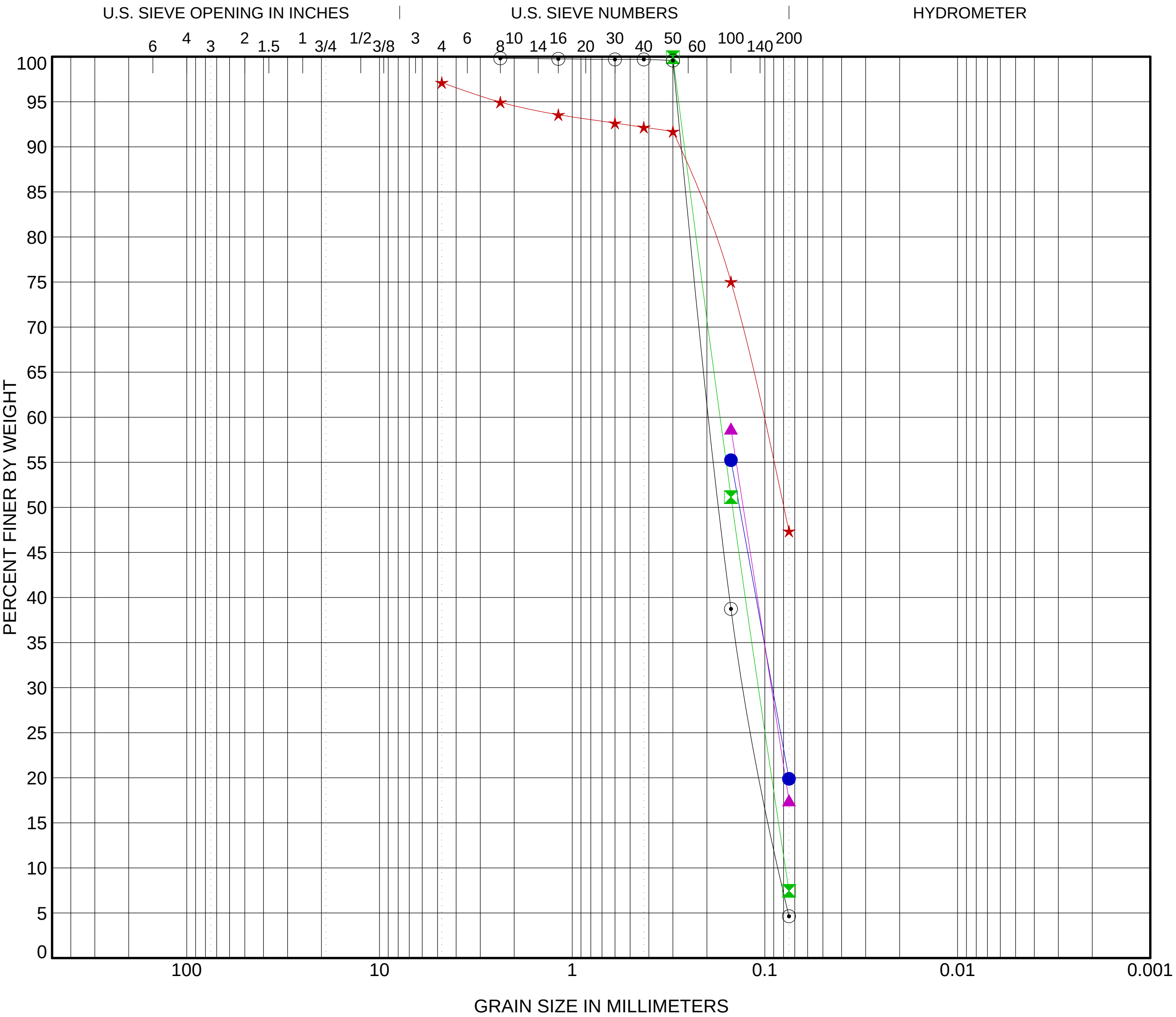
Elev. (ft)		LOG	Texas Cone Penetrometer	Strata Description	Triaxial Test Lateral Press. (psi)	Triaxial Test Deviator Stress (psi)	Properties MC	Properties LL	Properties PI	Properties Wet Den. (pcf)	Additional Remarks
				SAND, poorly-graded, loose, semi-moist, dark brown, with seaweeds (SP)			18				
5.0				SAND, poorly-graded, with silt, compact to slightly compact, brown to grayish-brown (SP-SM)			23				PI = Non-Plastic
	5		18 (6) 25 (6)								
							28				-200% = 8
	10		8 (6) 12 (6)								
							33				
	15		13 (6) 18 (6)								
							27				PI = Non-Plastic
	20		25 (6) 9 (6)								
							30				
	25		18 (6) 15 (6)								
							22				-200% = 7
-23.0	30		12 (6) 28 (6)								
	35										
	40										
	45										

Remarks:

The ground surface elevation was provided to us by Urban Engineering via electronic-mail attachment on Wednesday, March 14, 2012.

Any ground water elevation information provided on this boring log is representative of conditions existing on the day and for the specific location where this information was collected. The actual groundwater elevation may fluctuate due to time, climatic conditions, and/or construction activity.

Driller: JEDI Pumps **Logger: A. Rodriguez** **Organization: Raba Kistner Consultants, Inc.**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**32.5**	**SILTY SAND (SM)**					
⧗	**B-1**	**52.5**	**POORLY-GRADED SAND with SILT (SP-SM)**				**0.87**	**2.18**
▲	**B-2**	**27.5**	**SILTY SAND (SM)**					
★	**B-2**	**57.5**	**CLAYEY SAND (SC)**					
⊙	**B-3**	**17.5**	**POORLY-GRADED SAND (SP)**				**0.99**	**2.29**

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**32.5**	**0.15**		**0.091**		**0.0**	**35.4**	**19.9**	
⧗	**B-1**	**52.5**	**0.3**	**0.17**	**0.107**	**0.078**	**0.0**	**92.5**	**7.5**	
▲	**B-2**	**27.5**	**0.15**		**0.093**		**0.0**	**41.3**	**17.5**	
★	**B-2**	**57.5**	**4.75**	**0.103**			**0.0**	**49.8**	**47.4**	
⊙	**B-3**	**17.5**	**2.36**	**0.191**	**0.126**	**0.084**	**0.0**	**95.2**	**4.6**	

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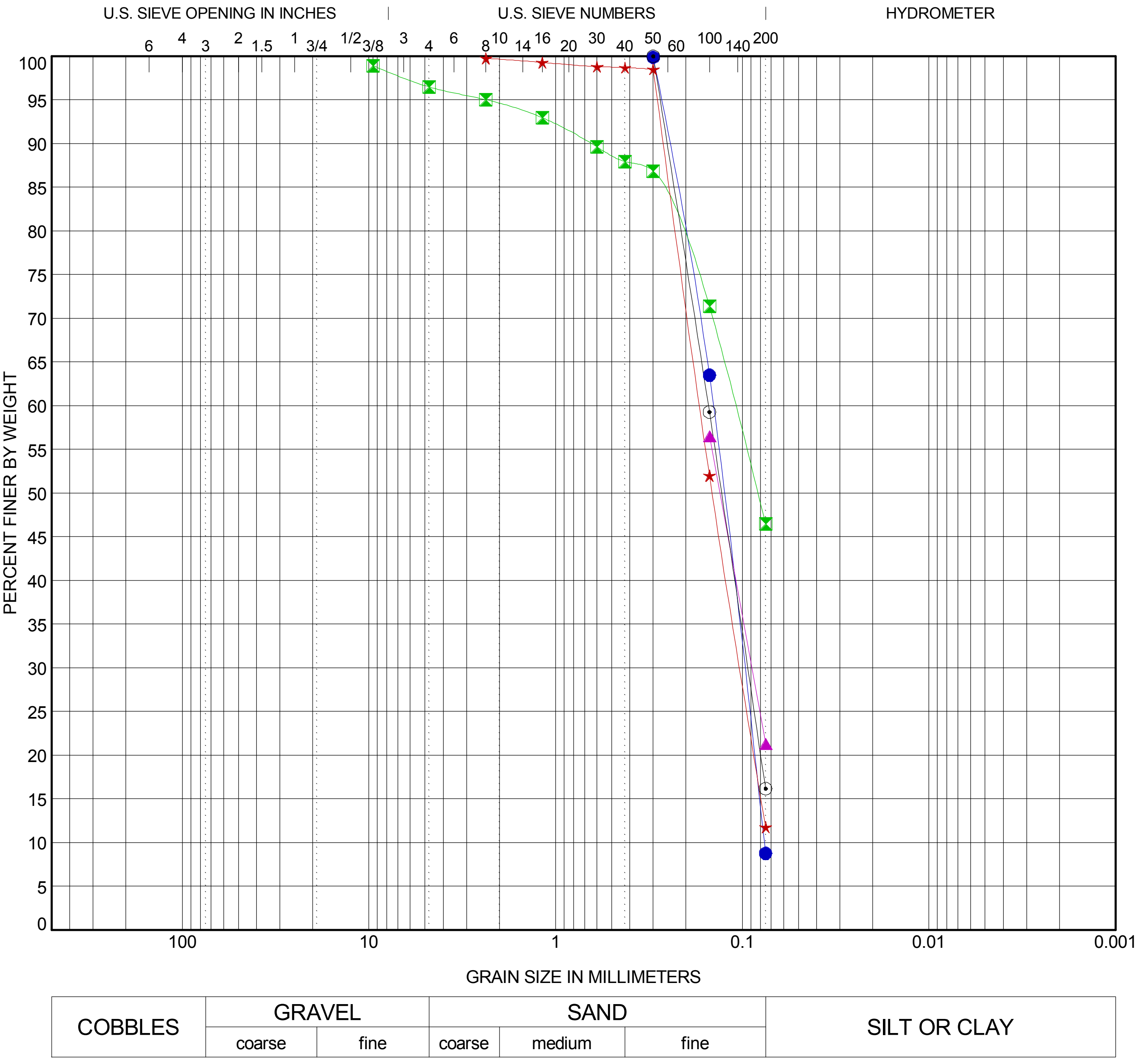
GRAIN SIZE DISTRIBUTION

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**B-3**	**47.5**	**POORLY-GRADED SAND with SILT (SP-SM)**				**0.88**	**1.88**
⧗	**B-3**	**57.5**	**CLAYEY SAND (SC)**					
▲	**B-4**	**32.5**	**CLAYEY SAND (SC)**					
★	**B-4**	**52.5**	**POORLY-GRADED SAND with SILT (SP-SM)**				**0.86**	**2.32**
⊙	**B-13**	**27.5**	**SILTY SAND (SM)**					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-3**	**47.5**	**0.3**	**0.144**	**0.098**	**0.076**	**0.0**	**91.1**	**8.8**	
⧗	**B-3**	**57.5**	**9.5**	**0.109**			**2.4**	**50.0**	**46.5**	
▲	**B-4**	**32.5**	**0.15**		**0.089**		**0.0**	**35.2**	**21.3**	
★	**B-4**	**52.5**	**2.36**	**0.169**	**0.103**		**0.0**	**88.0**	**11.8**	
⊙	**B-13**	**27.5**	**0.3**	**0.152**	**0.094**		**0.0**	**83.8**	**16.2**	

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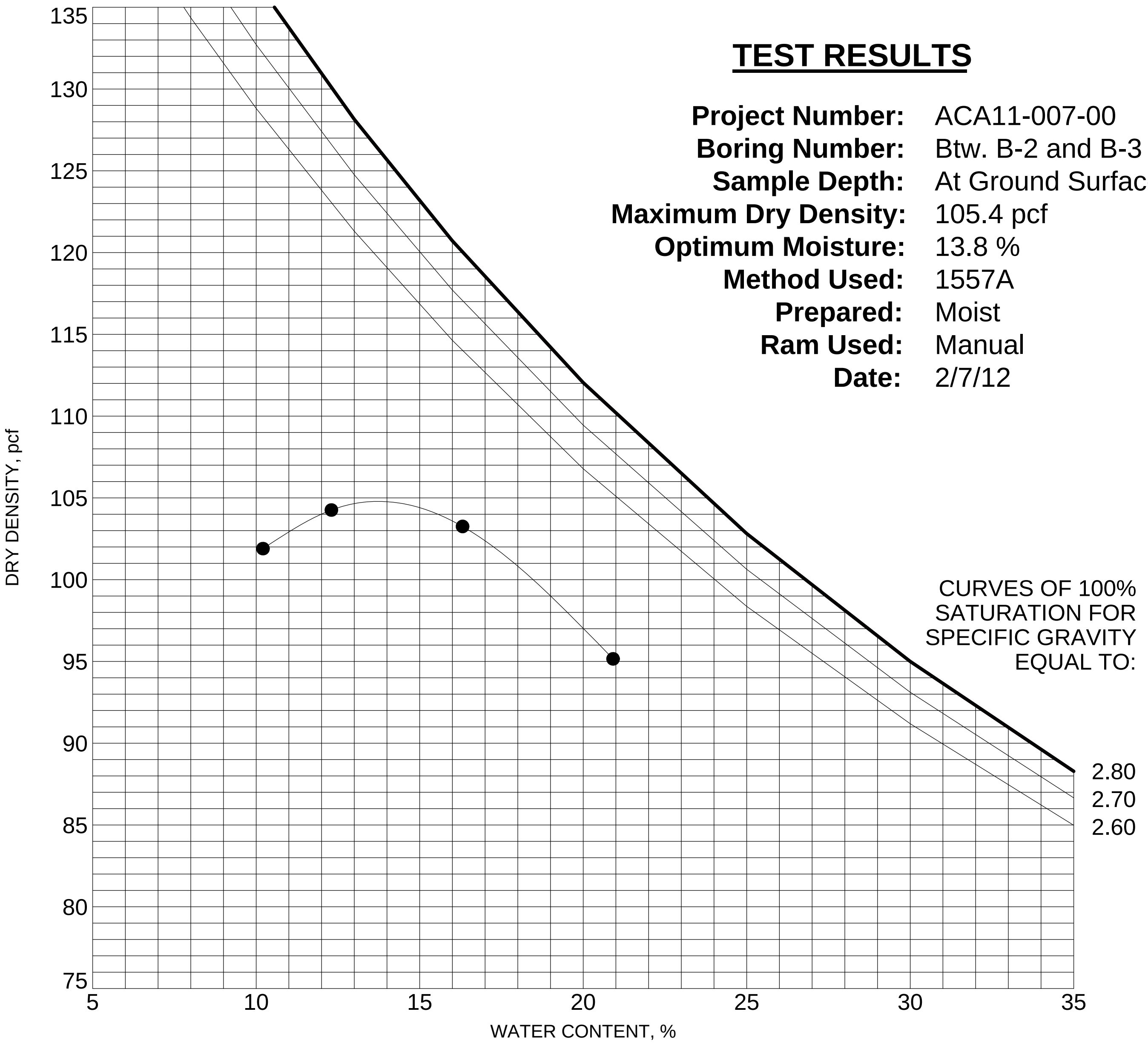
GRAIN SIZE DISTRIBUTION

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

FIGURE A-13

R-K GRAIN SIZE ACA11-007-00 TXDOT BORINGS.GPJ RKCI.GDT 3/14/12



TEST RESULTS

Project Number:	ACA11-007-00
Boring Number:	Btw. B-2 and B-3
Sample Depth:	At Ground Surface
Maximum Dry Density:	105.4 pcf
Optimum Moisture:	13.8 %
Method Used:	1557A
Prepared:	Moist
Ram Used:	Manual
Date:	2/7/12

CBR (California Bearing Ratio) of Laboratory-Compacted Soils (ASTM D 1889):

CBR:	19
% Swell:	1.3

Remolded at 97.6% of the maximum dry density determined above.

Soil Type:

Borrow Source Description:	Between Borings B-2 and B-3
Classification (ASTM D 2488):	POORLY-GRADED SAND (SP)
Classification (Visual):	POORLY-GRADED SAND

FIGURE A-14

MOISTURE-DENSITY RELATIONSHIP

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

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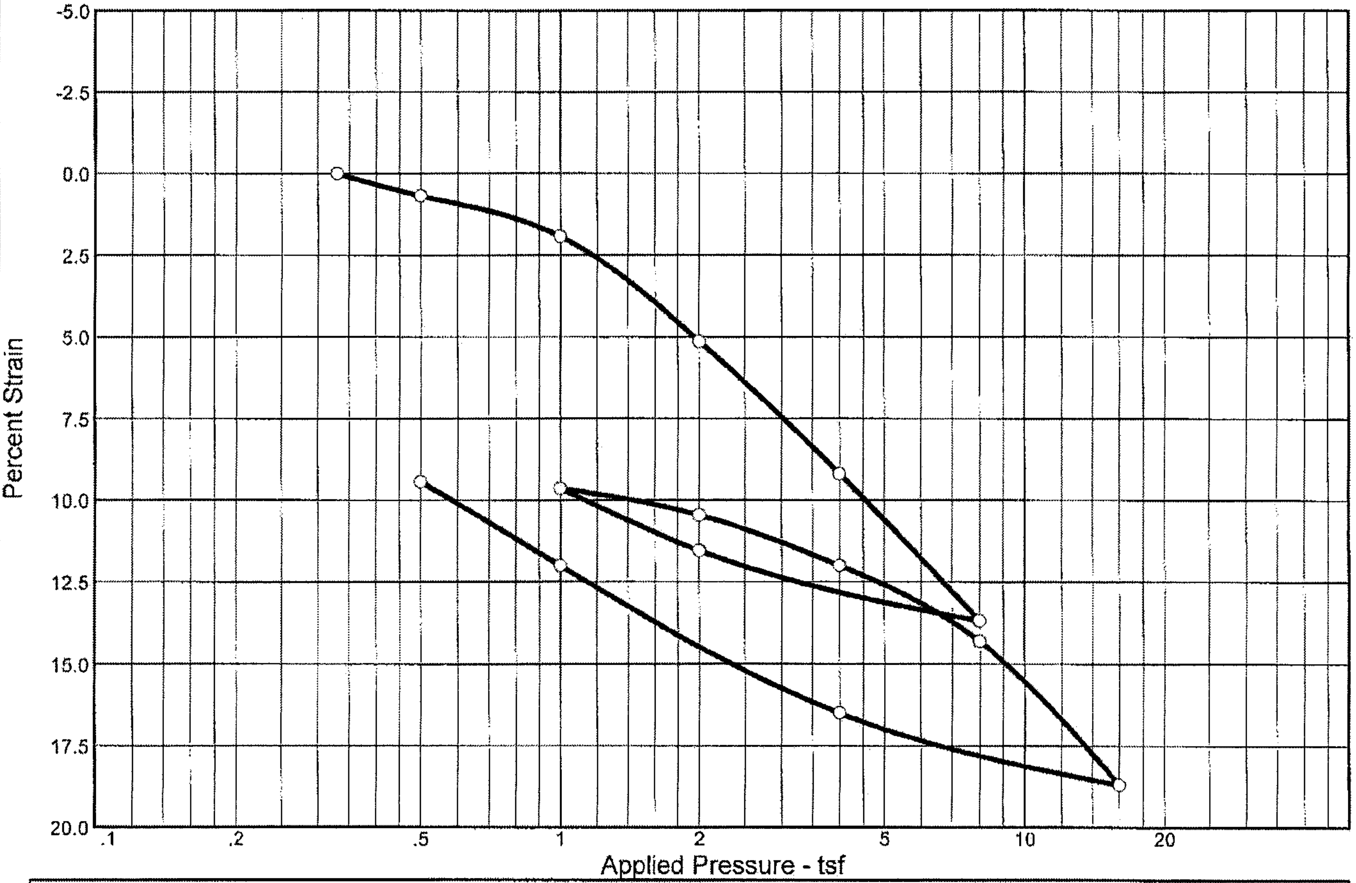
CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation											
No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
2	0.50	0.02		12	16.00	0.00					
3	1.00	0.02		13	4.00	0.01					
4	2.00	0.01		14	1.00	0.00					
5	4.00	0.00		15	0.50	0.00					
6	8.00	0.00									
7	2.00	0.01									
8	1.00	0.00									
9	2.00	0.01									
10	4.00	0.01									
11	8.00	0.01									

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture								
105.7 %	38.7 %	84.0	63	46	2.65	4.00	1.82	0.33	0.970

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy fat clay, bluish-brown	CH	

Project No. ACA11-007 **Client:** City of Corpus Christi

Project: Park Road 22 Bridge Project

Location: Boring 1 Sample 15 68ft

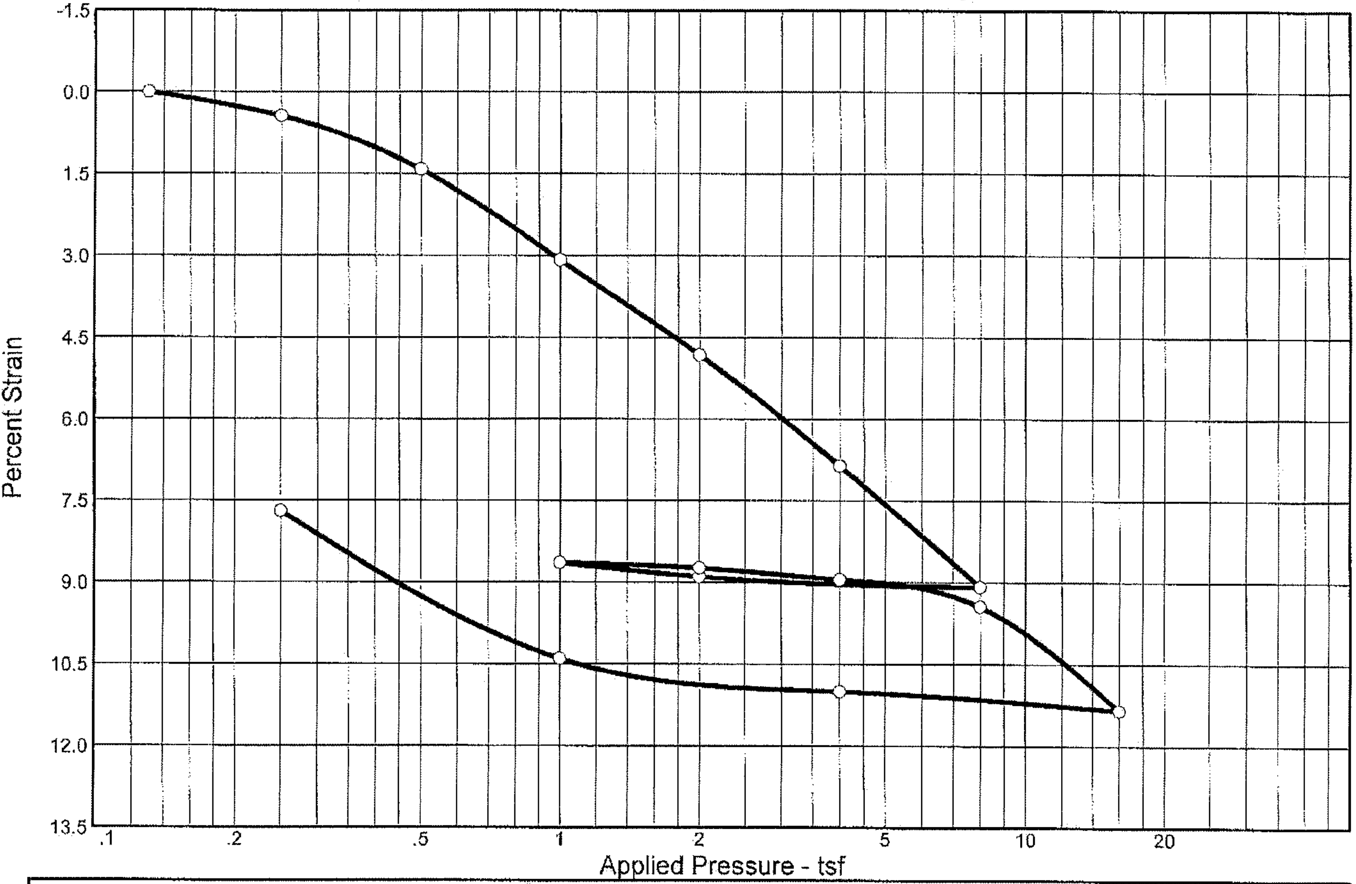
Remarks:

ASTM D2435
estimated specific gravity
Swell Pressure= 0.33 tsf

CONSOLIDATION TEST REPORT

RABA KISTNER CONSULTANTS, INC.

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation											
No.	Load (tsf)	C_V (ft.^{2}/day)	C_α	No.	Load (tsf)	C_V (ft.^{2}/day)	C_α	No.	Load (tsf)	C_V (ft.^{2}/day)	C_α
2	0.25	0.02		12	8.00	0.18					
3	0.50	0.02		13	16.00	0.16					
4	1.00	0.02		14	4.00	0.31					
5	2.00	0.02		15	1.00	0.13					
6	4.00	0.16		16	0.25	0.01					
7	8.00	0.06									
8	2.00	0.18									
9	1.00	0.04									
10	2.00	0.37									
11	4.00	0.37									

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_C (tsf)	C_C	Initial Void Ratio
Saturation	Moisture								
91.9 %	25.3 %	95.7	38	24	2.65	2.28	0.92	0.13	0.729

MATERIAL DESCRIPTION	USCS	AASHTO
Clayey sandy, bluish-brown	SC	

Project No. ACA11-007 **Client:** City of Corpus Christi

Project: Park Road 22 Bridge Project

Location: Boring 4 Sample 9 38ft

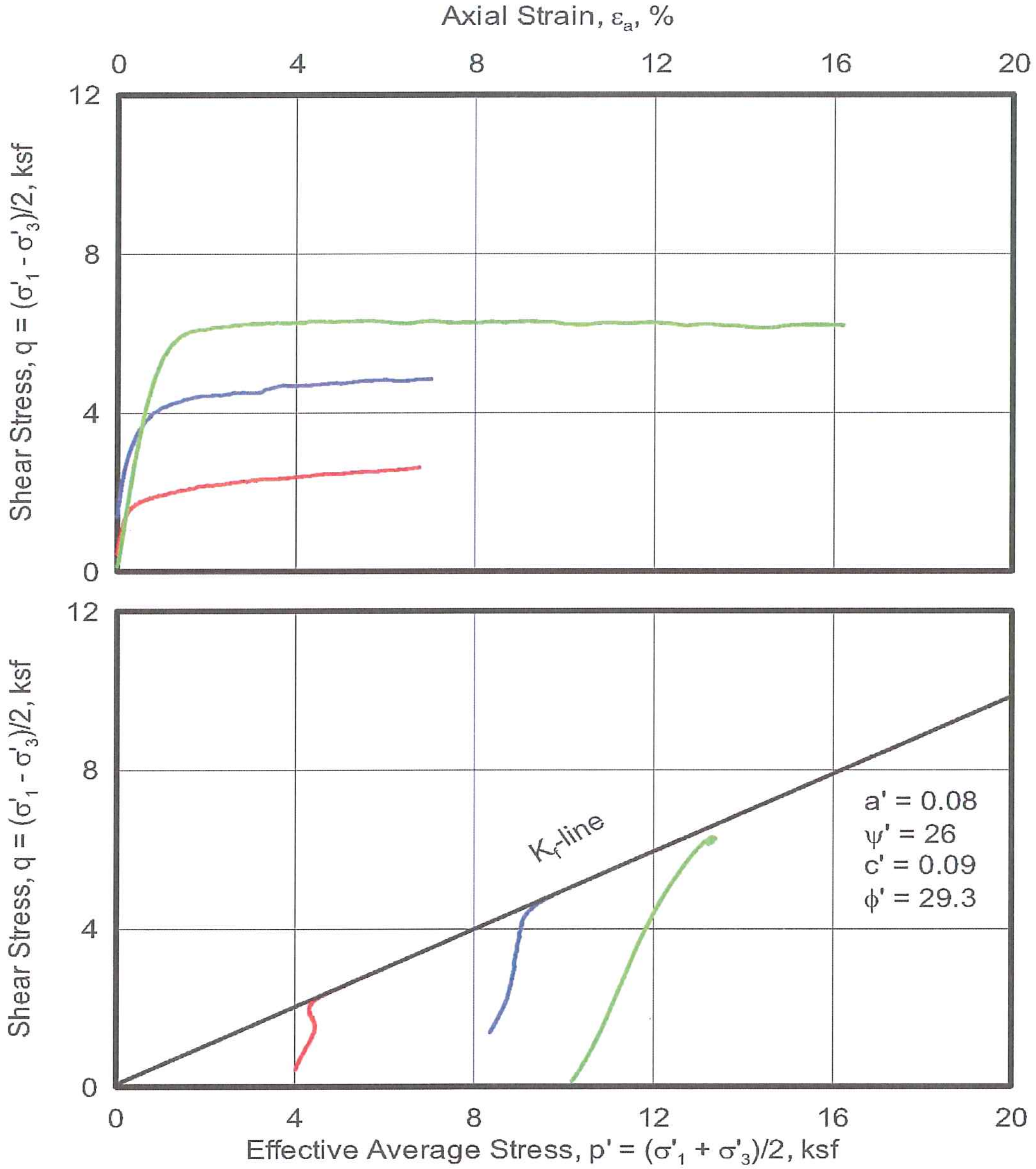
Remarks:

ASTM D2435
estimated specific gravity
Swell Pressure= 0.13 tsf

CONSOLIDATION TEST REPORT

RABA KISTNER CONSULTANTS, INC.

PROJECT ACA11-007-00



MULTI STAGE TRIAXIAL UNDRAINED COMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS PATH
BORING B-1, DEPTH 42 FEET

MATERIAL: SANDY LEAN CLAY (CL), bluish-brown
INITIAL WATER CONTENT: 25.6
INITIAL DRY UNIT WEIGHT: 96.6 pcf
INITIAL VOID RATIO: 0.78
SPECIFIC GRAVITY: 2.75 (assumed)

FINAL WATER CONTENT: 22
INITIAL DEGREE OF SATURATION: 90%
FINAL DEGREE OF SATURATION: 100%
LL = 30; PL = 15; PI = 15

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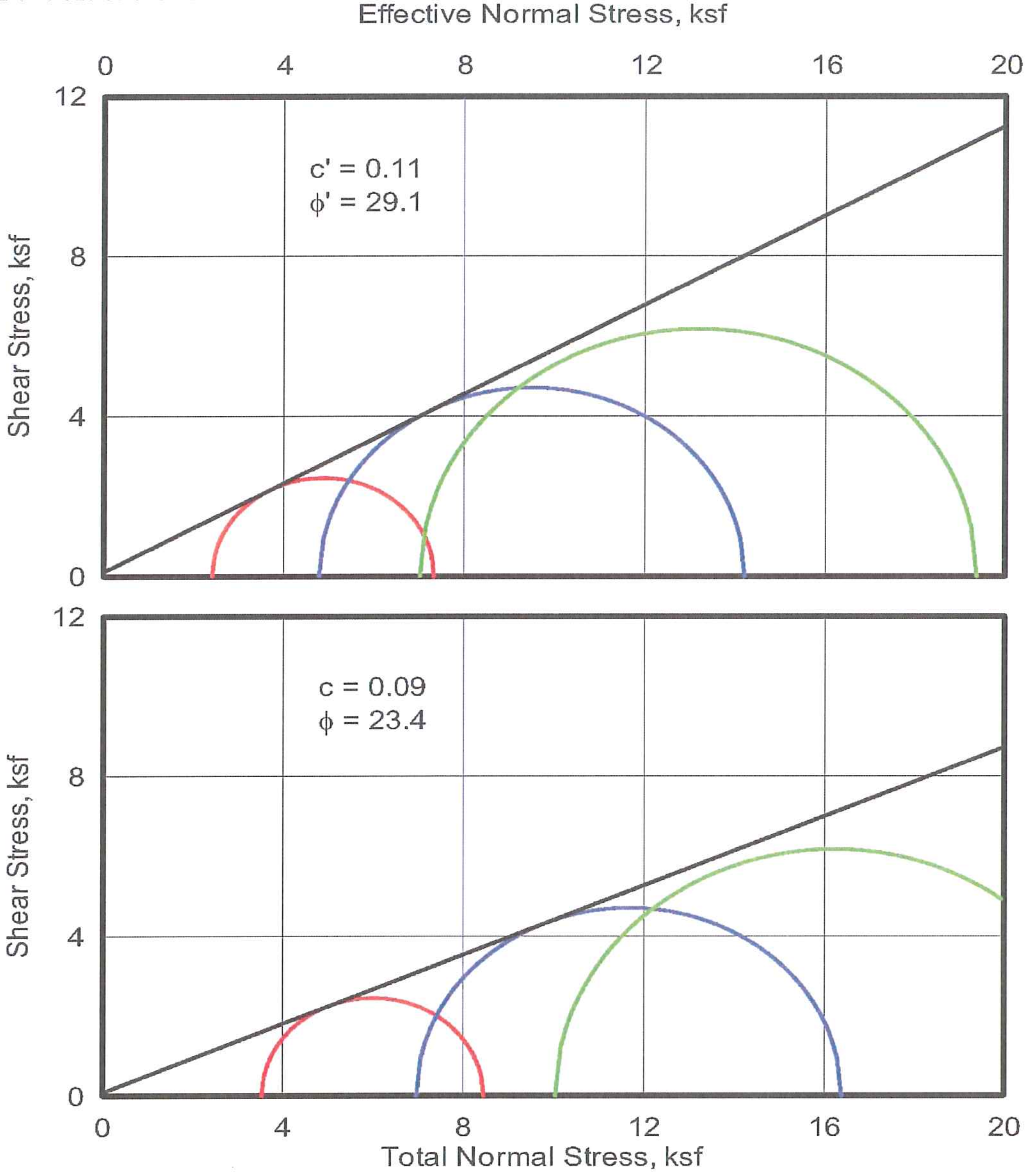
STRESS PATH

PARK ROAD 22 BRIDGE
(PROJECT NO. 6281)
CORPUS CHRISTI, TEXAS

PROJECT ACA11-007-00



MULTI STAGE TRIAXIAL UNDRAINED COMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS PATH
BORING B-1, DEPTH 42 FEET

MATERIAL: SANDY LEAN CLAY (CL), bluish-brown

INITIAL WATER CONTENT: 25.6	FINAL WATER CONTENT: 22
INITIAL DRY UNIT WEIGHT: 96.6 pcf	INITIAL DEGREE OF SATURATION: 90%
INITIAL VOID RATIO: 0.78	FINAL DEGREE OF SATURATION: 100%
SPECIFIC GRAVITY: 2.75 (assumed)	LL = 30; PL = 15; PI = 15

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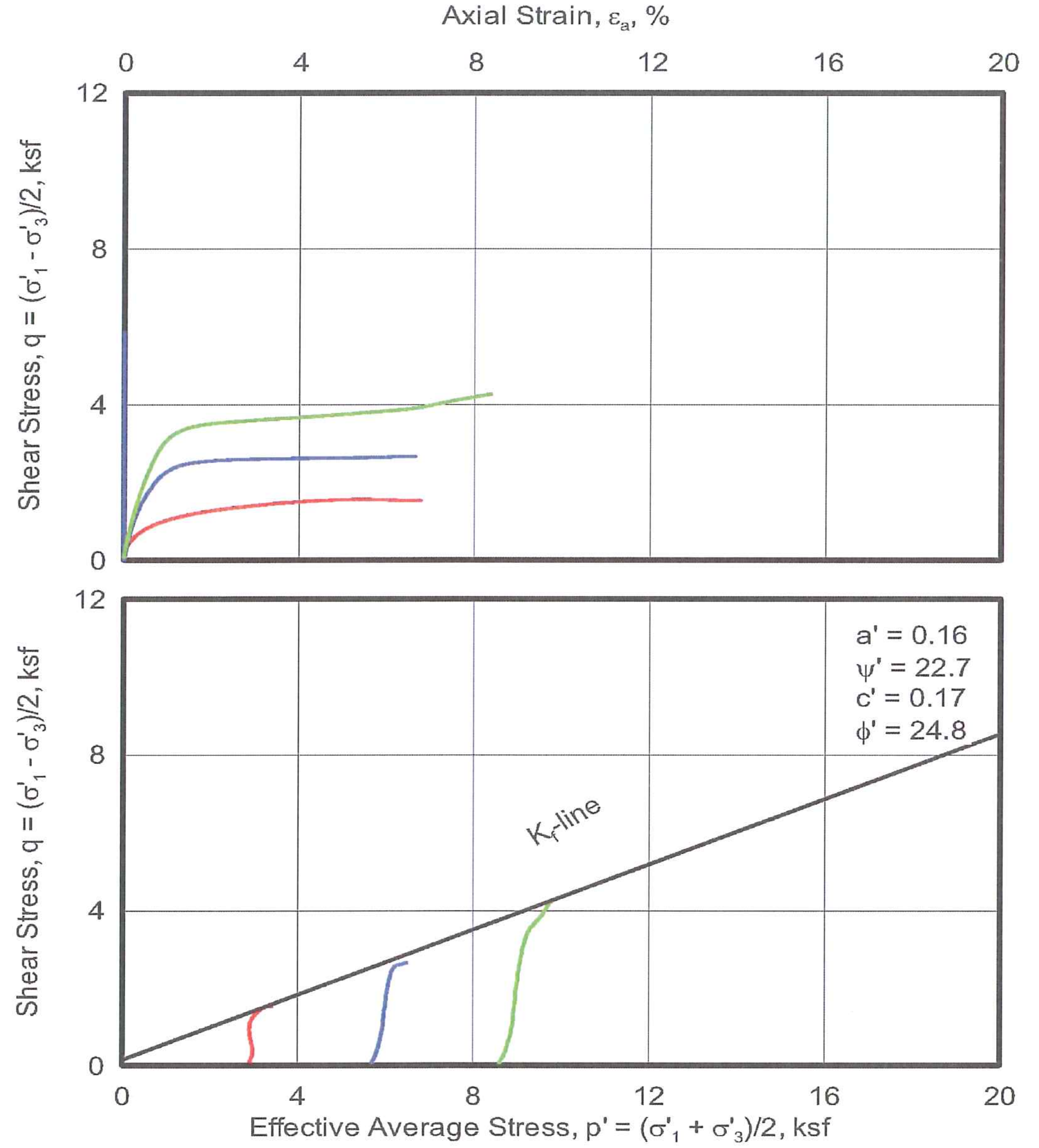
MOHR'S CIRCLES

PARK ROAD 22 BRIDGE
(PROJECT NO. 6281)
CORPUS CHRISTI, TEXAS

PROJECT ACA11-007-00



MULTI STAGE TRIAXIAL UNDRAINED COMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS PATH
BORING B-1, DEPTH 62.5 FEET

MATERIAL: FAT CLAY w/ SAND (CH), bluish-brown to grayish-brown
INITIAL WATER CONTENT: 25.6
INITIAL DRY UNIT WEIGHT: 94 pcf
INITIAL VOID RATIO: 0.83
SPECIFIC GRAVITY: 2.75 (assumed)

FINAL WATER CONTENT: 24.8
INITIAL DEGREE OF SATURATION: 85%
FINAL DEGREE OF SATURATION: 100%
LL = 66; PL = 22; PI = 44

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STRESS PATH

PARK ROAD 22 BRIDGE
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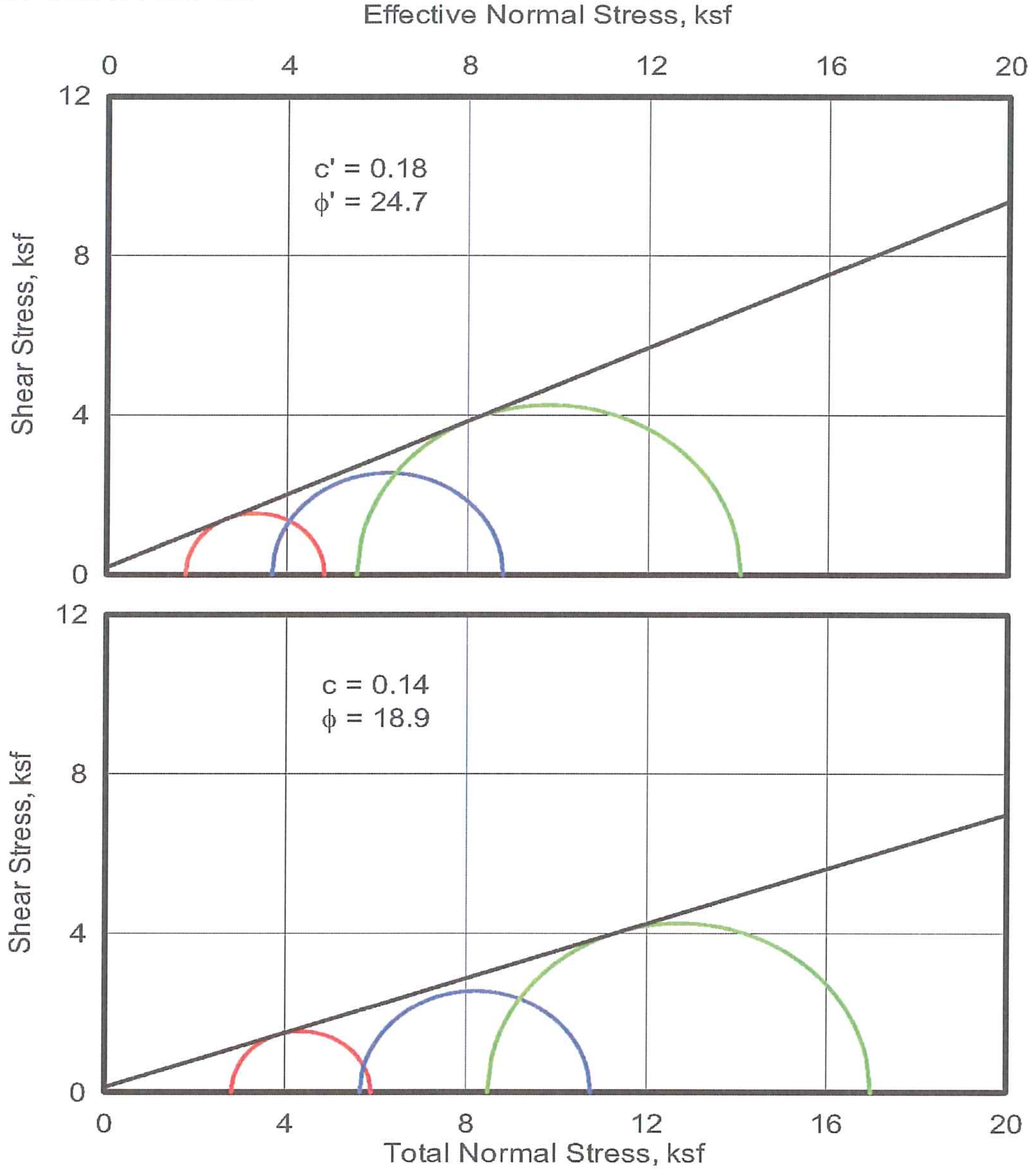
CORPUS CHRISTI, TEXAS

PROJECT ACA11-007-00



MULTI STAGE TRIAXIAL UNDRAINED COMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS PATH
BORING B-1, DEPTH 62.5 FEET

MATERIAL: FAT CLAY w/ SAND (CH), bluish-brown to grayish-brown

INITIAL WATER CONTENT: 25.6	FINAL WATER CONTENT: 24.8
INITIAL DRY UNIT WEIGHT: 94 pcf	INITIAL DEGREE OF SATURATION: 85%
INITIAL VOID RATIO: 0.83	FINAL DEGREE OF SATURATION: 100%
SPECIFIC GRAVITY: 2.75 (assumed)	LL = 66; PL = 22; PI = 44

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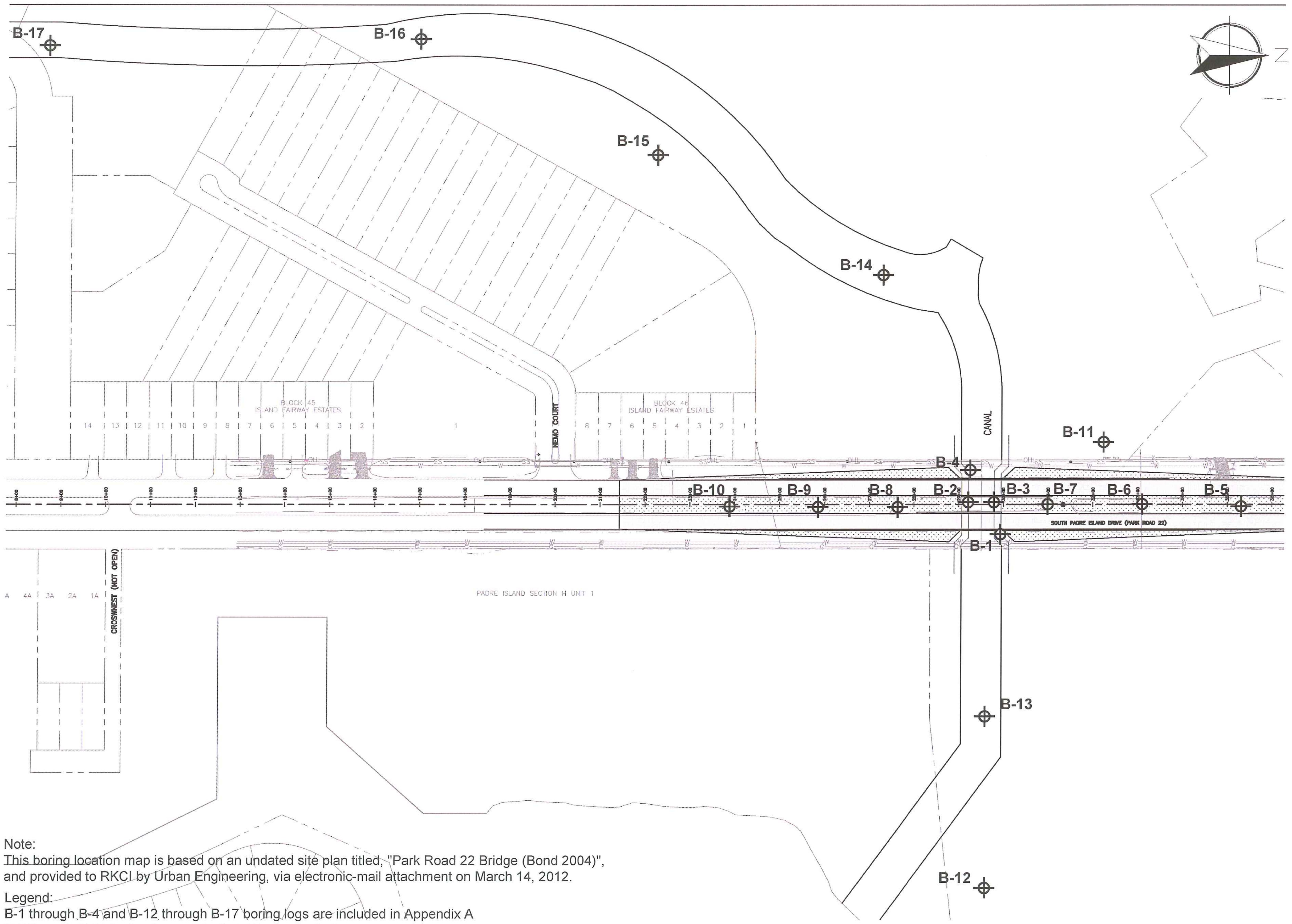
MOHR'S CIRCLES

PARK ROAD 22 BRIDGE
(PROJECT NO. 6281)
CORPUS CHRISTI, TEXAS

APPENDIX B



Note:
This boring location map is based on an undated site plan titled, "Park Road 22 Bridge (Bond 2004)", and provided to RKCI by Urban Engineering, via electronic-mail attachment on March 14, 2012.

Legend:
B-1 through B-4 and B-12 through B-17 boring logs are included in Appendix A
B-5 through B-11 boring logs are included in Appendix B

O:\Active Projects\McAllen\2011\ACA – Corpus Christi\ACA11-007-00 Park Road 22\Drawings\Figure 1 – 11x17.dwg

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BORING LOCATION MAP
PARK ROAD 22 BRIDGE PROJECT
ALONG SOUTH PADRE ISLAND DRIVE
CORPUS CHRISTI, NUECES COUNTY, TEXAS

REVISIONS:

No.	DATE	DESCRIPTION

PROJECT No.:
ACA11-007-00

ISSUE DATE:	03-14-12
DRAWN BY:	NES
CHECKED BY:	JLP
REVIEWED BY:	KML

FIGURE
B-1

LOG OF BORING NO. B-5

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

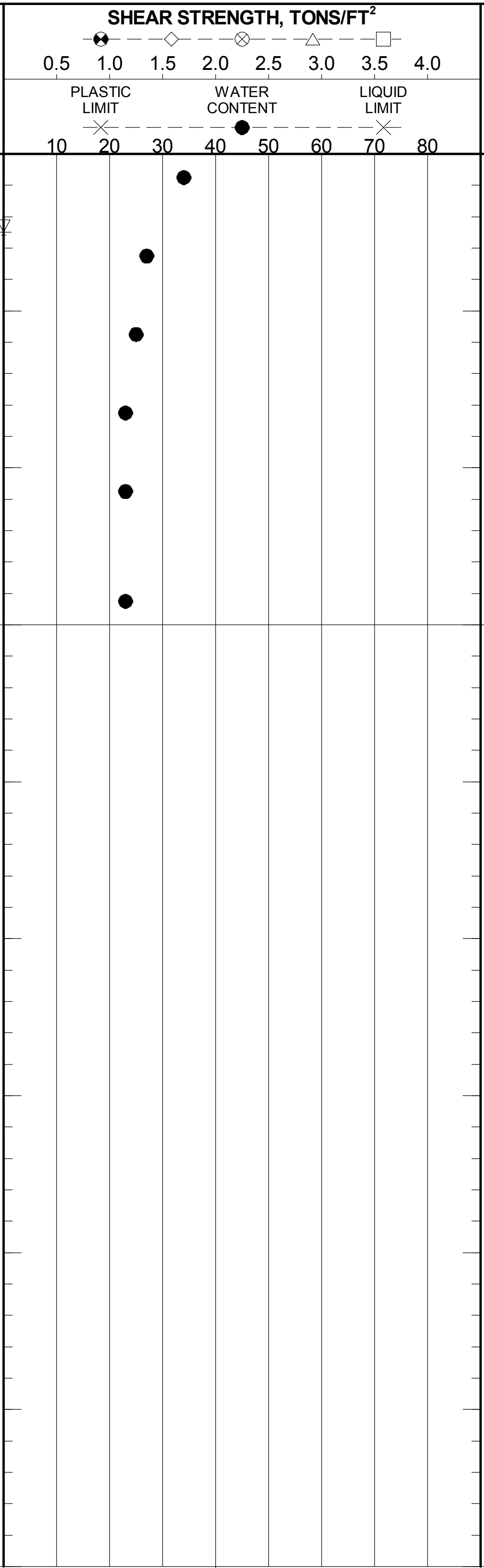
Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 17113300.19000; E 1398136.39000

SURFACE ELEVATION: 2.09 ft

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			POORLY-GRADED SAND (SP) medium dense to loose to medium dense, dark grayish-brown	12		NP	
				14			1
5				7			
			- becomes light grayish-brown in color below a depth of about 7 ft	7			
10				18			
				21			
15			Boring terminated at a depth of about 15 ft.				



NOTES:
During the drilling operations, groundwater was encountered at a depth of about 2.5 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 4 ft, and the boring caved-in to a depth of about 6 ft.

The ground surface elevation shown above was provided to us by Urban Engineering, via electronic-mail attachment on Wednesday, March 14, 2012.

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	2.5 ft	**PROJ. No.:**	ACA11-007-00
DATE DRILLED:	1/27/2012	**DATE MEASURED:**	1/27/2012	**FIGURE:**	B-2

LOG OF BORING NO. B-6

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

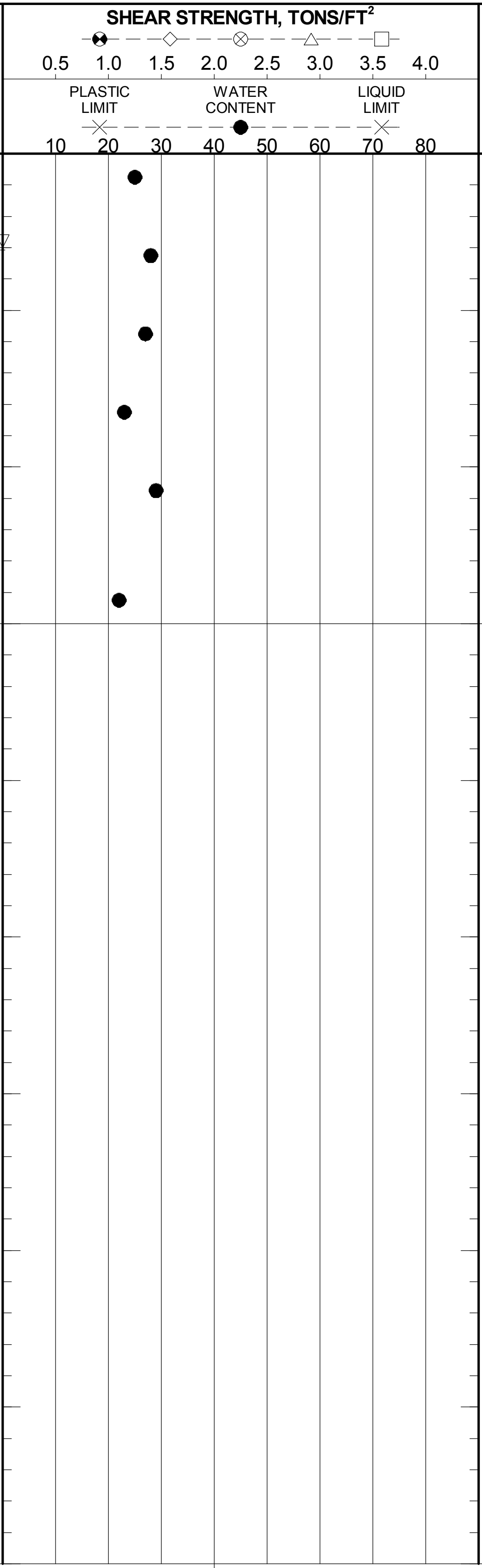
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 17113080.99000; E 1398133.58000

SURFACE ELEVATION: 2.22 ft

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			POORLY-GRADED SAND (SP) medium dense to loose, dark grayish-brown	20			3
				14	▽	NP	
5				4			
			- becomes light grayish-brown in color below a depth of about 7 ft	9			
10				6			
				7			
15			Boring terminated at a depth of about 15 ft.				



NOTES:
During the drilling operations, groundwater was encountered at a depth of about 3 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 4.5 ft, and the boring caved-in to a depth of about 6 ft.

The ground surface elevation shown above was provided to us by Urban Engineering, via electronic-mail attachment on Wednesday, March 14, 2012.

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	3 ft	**PROJ. No.:**	ACA11-007-00
DATE DRILLED:	1/27/2012	**DATE MEASURED:**	1/27/2012	**FIGURE:**	B-3

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

LOG OF BORING NO. B-7

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

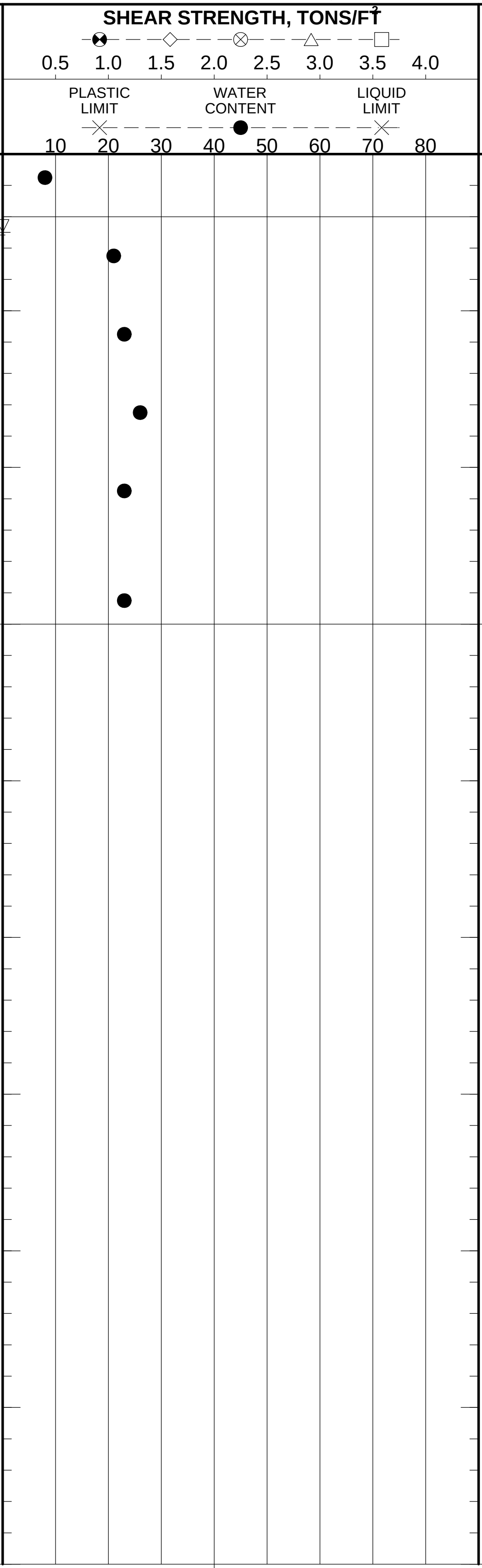
DRILLING METHOD: Hollow Stem Auger

LOCATION: N 17112906.39000; E 1398130.00000 *

SURFACE ELEVATION: Approx. 2.5 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
POORLY-GRADED SAND (SP) medium dense, dark grayish-brown, with seaweeds	30			
SILTY SAND (SM) dense to loose to medium dense, light grayish-brown	32			
	24		NP	
	9			13
	22			
	23			

Boring terminated at a depth of about 15 ft.



DEPTH, FT: 5, 10, 15, 20, 25, 30, 35, 40

NOTES:
During the drilling operations, groundwater was encountered at a depth of about 2.5 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 4.5 ft, and the boring caved-in to a depth of about 6 ft.

The ground surface elevation shown above was provided to us by Urban Engineering, via electronic-mail attachment on Wednesday, March 14, 2012.

* The boring was moved and drilled at about 30 ft south of its originally staked location.

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	2.5 ft	**PROJ. No.:**	ACA11-007-00
DATE DRILLED:	1/27/2012	**DATE MEASURED:**	1/27/2012	**FIGURE:**	B-4

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

LOG OF BORING NO. B-8

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

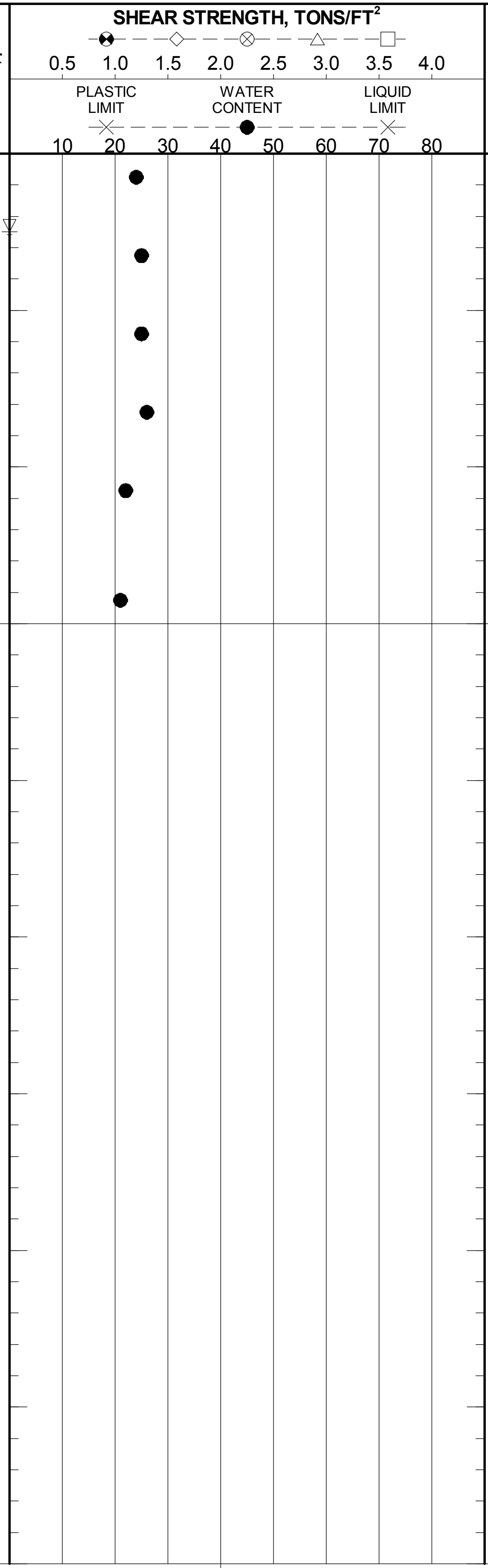
Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 17112534.92000; E 1398128.27000

SURFACE ELEVATION: 2.14 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	POORLY-GRADED SAND (SP) loose to medium dense to loose to medium dense, dark grayish-brown	5		NP	
		11			3
5		7			
	- becomes light grayish-brown in color below a depth of about 7 ft	12			
10		29			
		11			
15	Boring terminated at a depth of about 15 ft.				
20	NOTES: During the drilling operations, groundwater was encountered at a depth of about 2.5 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 4 ft, and the boring caved-in to a depth of about 7 ft.				
25	The ground surface elevation shown above was provided to us by Urban Engineering, via electronic-mail attachment on Wednesday, March 14, 2012.				
30					
35					
40					



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	2.5 ft	**PROJ. No.:**	ACA11-007-00
DATE DRILLED:	1/27/2012	**DATE MEASURED:**	1/27/2012	**FIGURE:**	B-5

LOG OF BORING NO. B-9

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

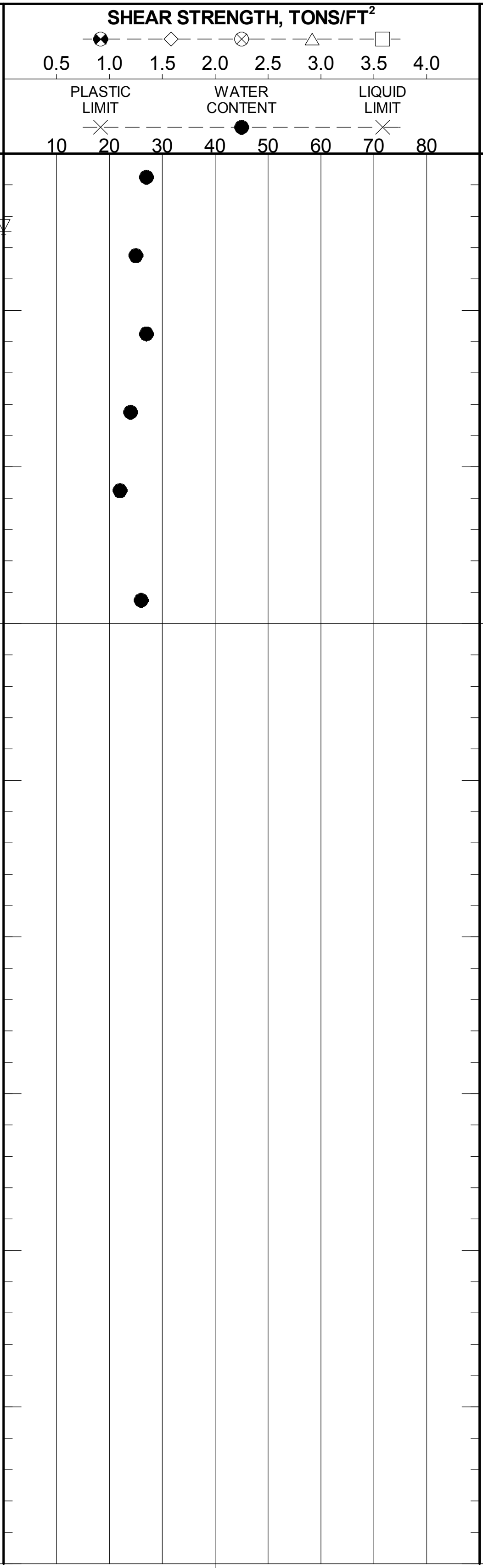
Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 17112354.99000; E 1398125.18000

SURFACE ELEVATION: 1.71 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0–1.5	**POORLY-GRADED SAND with SILT (SP-SM)** loose to medium dense to loose, dark grayish-brown	5			2
2.5–4		13		NP	
5–6.5		9			
7.5–9	- becomes light grayish-brown in color below a depth of about 7 ft	10			
10–11.5		7			
13.5–15		9			
15	Boring terminated at a depth of about 15 ft.				



NOTES:

During the drilling operations, groundwater was encountered at a depth of about 2.5 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 4.5 ft, and the boring caved-in to a depth of about 7 ft.

The ground surface elevation shown above was provided to us by Urban Engineering, via electronic-mail attachment on Wednesday, March 14, 2012.

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	2.5 ft	**PROJ. No.:**	ACA11-007-00
DATE DRILLED:	1/27/2012	**DATE MEASURED:**	1/27/2012	**FIGURE:**	B-6

LOG OF BORING NO. B-10

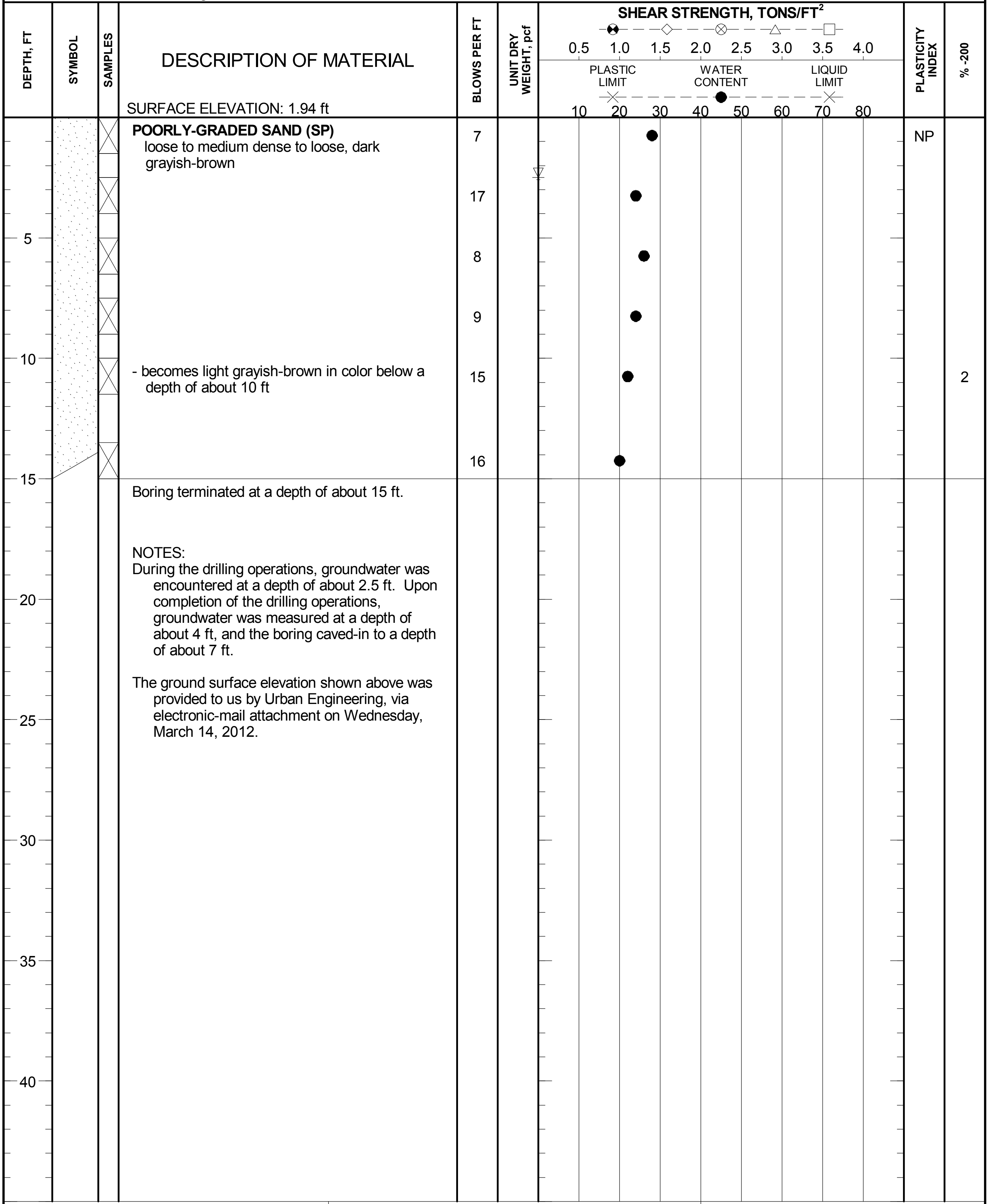
Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

Raba Kistner
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 17112163.46000; E 1398120.39000

SURFACE ELEVATION: 1.94 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
POORLY-GRADED SAND (SP) loose to medium dense to loose, dark grayish-brown	7		NP	
	17			
	8			
	9			
- becomes light grayish-brown in color below a depth of about 10 ft	15			2
	16			

Boring terminated at a depth of about 15 ft.

NOTES:

During the drilling operations, groundwater was encountered at a depth of about 2.5 ft. Upon completion of the drilling operations, groundwater was measured at a depth of about 4 ft, and the boring caved-in to a depth of about 7 ft.

The ground surface elevation shown above was provided to us by Urban Engineering, via electronic-mail attachment on Wednesday, March 14, 2012.

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	2.5 ft	**PROJ. No.:**	ACA11-007-00
DATE DRILLED:	1/27/2012	**DATE MEASURED:**	1/27/2012	**FIGURE:**	B-7

LOG OF BORING NO. B-11

Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

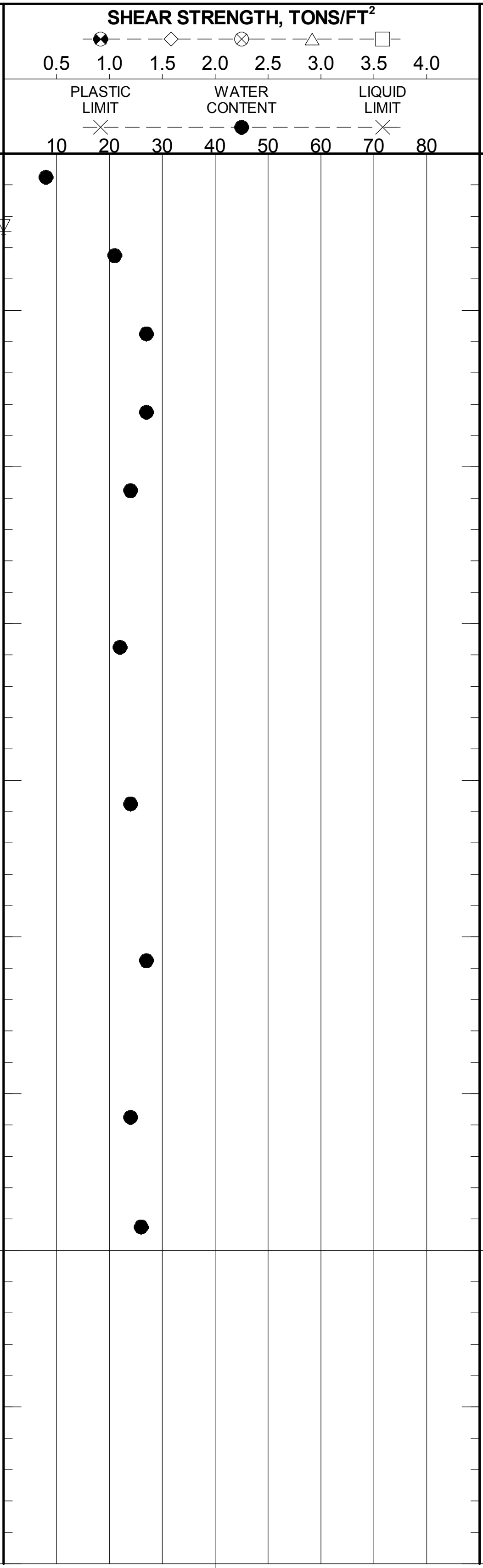
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem & Mud Rotary

LOCATION: N 17112998.23000; E 1397992.04000

SURFACE ELEVATION: 3.95 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	**POORLY-GRADED SAND (SP)** loose to medium dense, brown	7			
2.5		20			2
5	- becomes dark brown to grayish-brown in color below a depth of about 5 ft	11		NP	
7.5		12			
10		8			
15		27		NP	
20		12			1
25		13			
30	- becomes light grayish-brown in color below a depth of about 30 ft	23			
33.5		10			
35	Boring terminated at a depth of about 35 ft.				



NOTES:
During the drilling operations, groundwater was encountered at a depth of about 2.5 ft. Upon completion of the drilling operations, the boring remained uncaved.

The ground surface elevation shown above was provided to us by Urban Engineering, via electronic-mail attachment on Wednesday, March 14, 2012.

DEPTH DRILLED: 35.0 ft
DATE DRILLED: 1/30/2012
DEPTH TO WATER: 2.5 ft
DATE MEASURED: 1/30/2012
PROJ. No.: ACA11-007-00
FIGURE: B-8

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

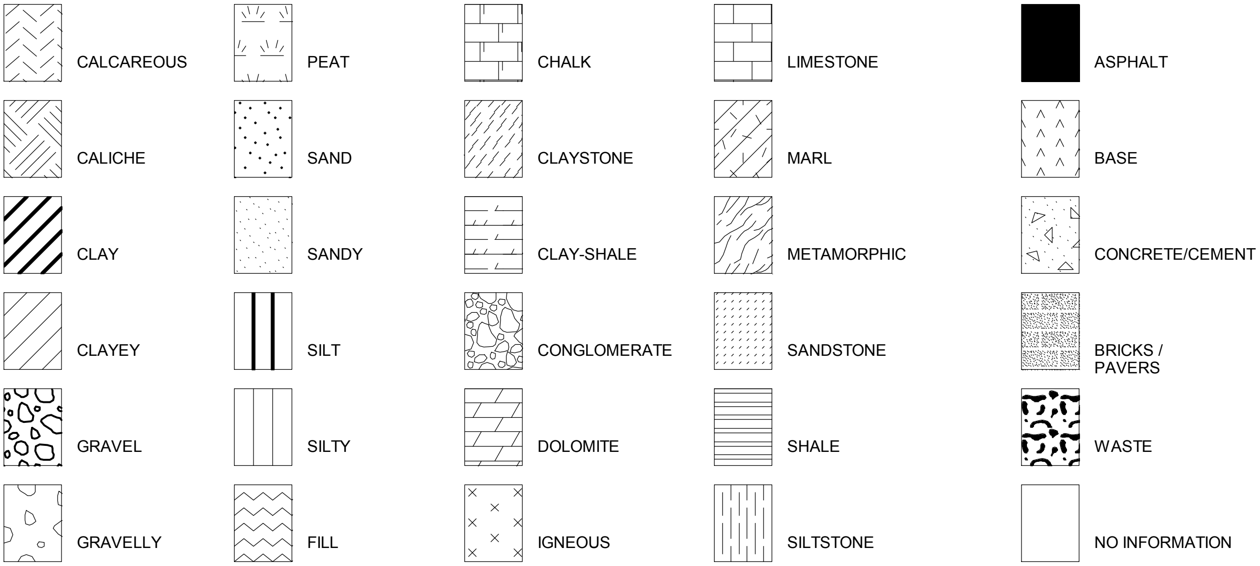
KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

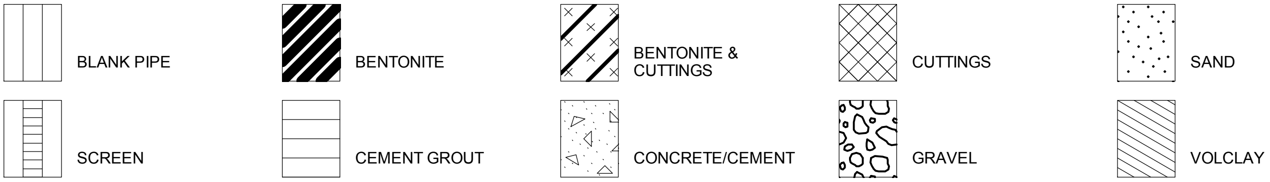
SOIL TERMS

ROCK TERMS

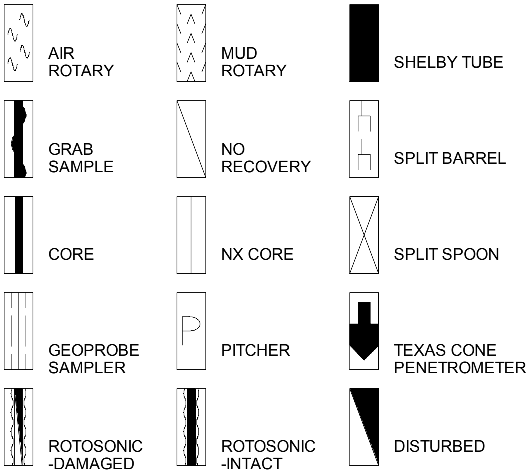
OTHER



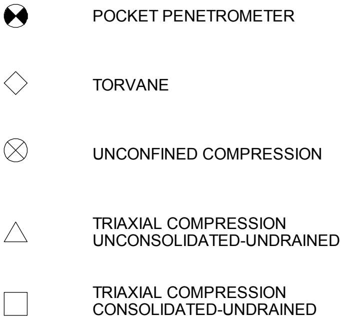
WELL CONSTRUCTION AND PLUGGING MATERIALS



SAMPLE TYPES



STRENGTH TEST TYPES



NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. ACA11-007-00

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

RELATIVE DENSITY

Penetration Resistance Blows per ft	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

COHESIVE STRENGTH

Resistance Blows per ft	Consistency	Cohesion TSF
0 - 2	Very Soft	0 - 0.125
2 - 4	Soft	0.125 - 0.25
4 - 8	Firm	0.25 - 0.5
8 - 15	Stiff	0.5 - 1.0
15 - 30	Very Stiff	1.0 - 2.0
> 30	Hard	> 2.0

PLASTICITY

Plasticity Index	Degree of Plasticity
0 - 5	None
5 - 10	Low
10 - 20	Moderate
20 - 40	Plastic
> 40	Highly Plastic

ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
El = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARRELL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

<u>NOTE:</u> To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

FILE NAME: ACA11-007-00STANDARD BORINGS.GPJ 3/14/2012

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-5	0.0 to 1.5	12	34	CNBD	CNBD	NP	SP				
	2.5 to 4.0	14	27						1		
	5.0 to 6.5	7	25								
	7.5 to 9.0	7	23								
	10.0 to 11.5	18	23								
	13.5 to 15.0	21	23								
B-6	0.0 to 1.5	20	25						3		
	2.5 to 4.0	14	28	CNBD	CNBD	NP	SP				
	5.0 to 6.5	4	27								
	7.5 to 9.0	9	23								
	10.0 to 11.5	6	29								
	13.5 to 15.0	7	22								
B-7	0.0 to 1.5	30	8								
	2.5 to 4.0	32	21								
	5.0 to 6.5	24	23	CNBD	CNBD	NP	SM				
	7.5 to 9.0	9	26						13		
	10.0 to 11.5	22	23								
	13.5 to 15.0	23	23								
B-8	0.0 to 1.5	5	24	CNBD	CNBD	NP	SP				
	2.5 to 4.0	11	25						3		
	5.0 to 6.5	7	25								
	7.5 to 9.0	12	26								
	10.0 to 11.5	29	22								
	13.5 to 15.0	11	21								
B-9	0.0 to 1.5	5	27						2		
	2.5 to 4.0	13	25	CNBD	CNBD	NP	SP-SM				
	5.0 to 6.5	9	27								
	7.5 to 9.0	10	24								
	10.0 to 11.5	7	22								
	13.5 to 15.0	9	26								
B-10	0.0 to 1.5	7	28	CNBD	CNBD	NP	SP				
	2.5 to 4.0	17	24								
	5.0 to 6.5	8	26								
	7.5 to 9.0	9	24								
	10.0 to 11.5	15	22						2		
	13.5 to 15.0	16	20								
B-11	0.0 to 1.5	7	8								
	2.5 to 4.0	20	21						2		
	5.0 to 6.5	11	27	CNBD	CNBD	NP	SP				

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial CNBD = Cound Not Be Determined NP = Non-Plastic

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Park Road 22 Bridge Project
Along South Padre Island Drive
Corpus Christi, Nueces County, Texas

FILE NAME: ACA11-007-00STANDARD BORINGS.GPJ 3/14/2012

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-11	7.5 to 9.0	12	27								
	10.0 to 11.5	8	24								
	15.0 to 16.5	27	22	CNBD	CNBD	NP	SP				
	20.0 to 21.5	12	24						1		
	25.0 to 26.5	13	27								
	30.0 to 31.5	23	24								
	33.5 to 35.0	10	26								

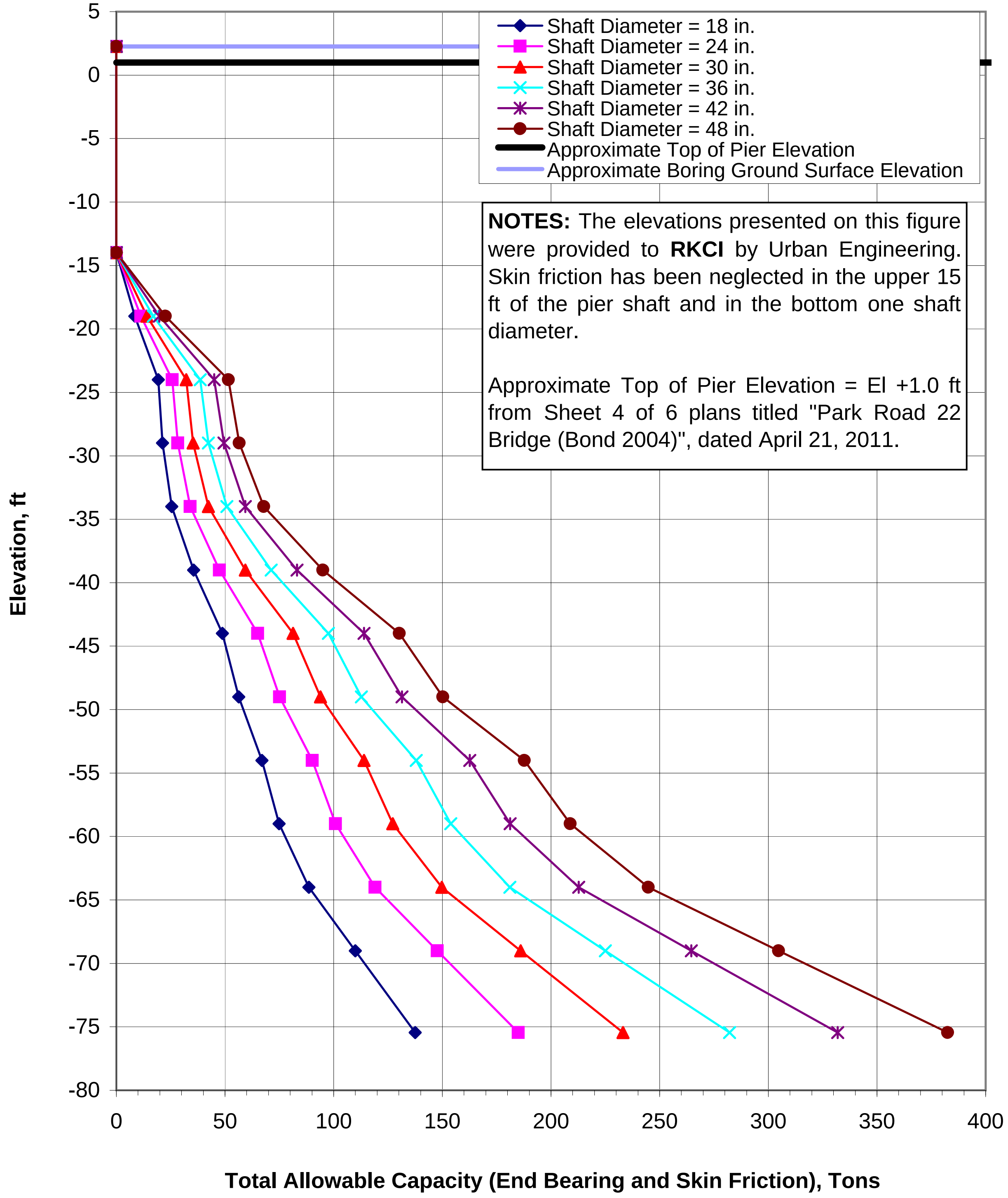
PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial CNBD = Cound Not Be Determined NP = Non-Plastic

APPENDIX C

DRILLED PIER AXIAL CAPACITY CURVE

Straight Shaft Piers
Park Road 22 (Proposed Bridge Borings)
Corpus Christi, Texas
East Abutment (Boring B-1)



NOTES: The elevations presented on this figure were provided to **RKCI** by Urban Engineering. Skin friction has been neglected in the upper 15 ft of the pier shaft and in the bottom one shaft diameter.

Approximate Top of Pier Elevation = El +1.0 ft from Sheet 4 of 6 plans titled "Park Road 22 Bridge (Bond 2004)", dated April 21, 2011.

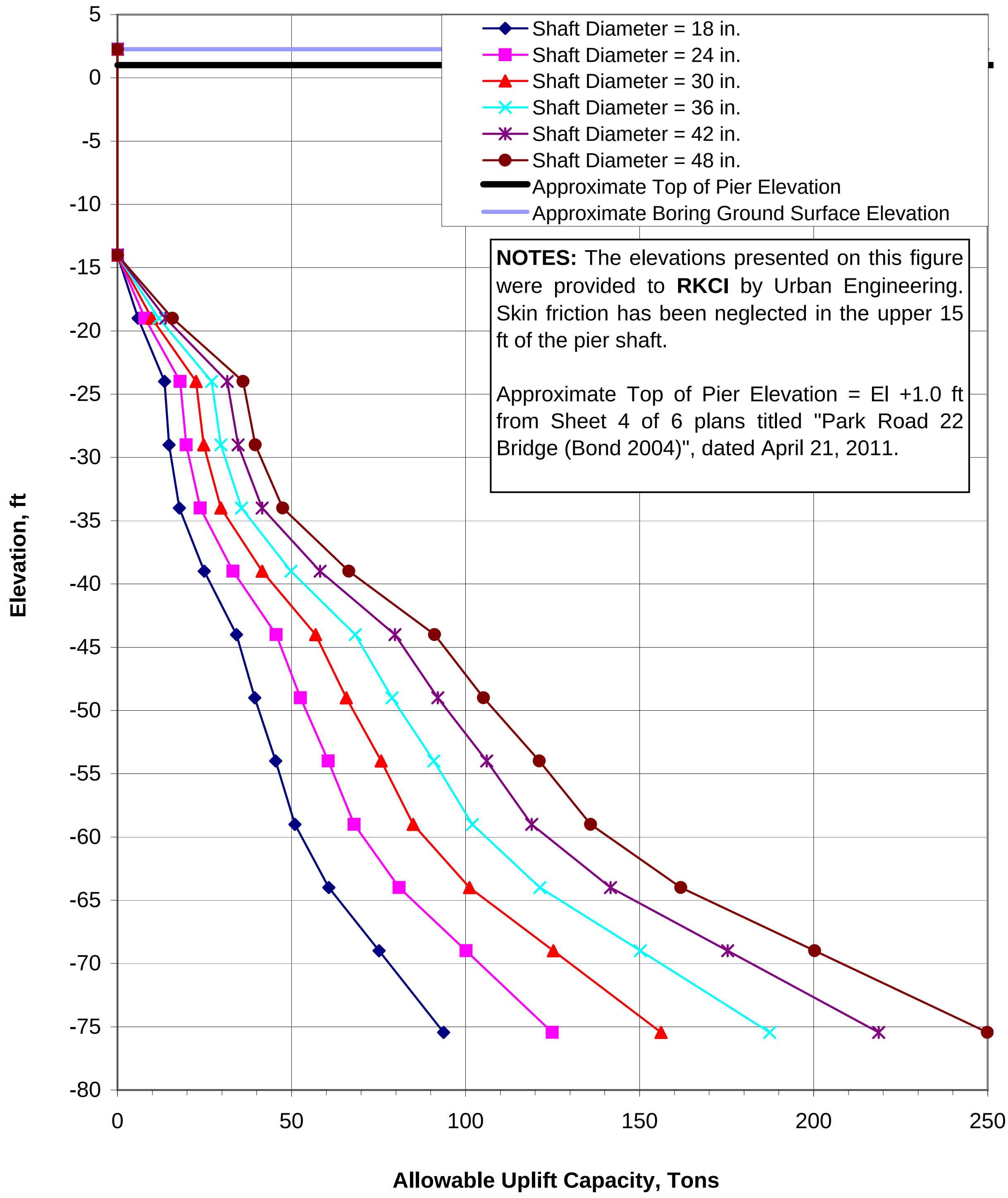
DRILLED PIER UPLIFT CAPACITY CURVE

Straight Shaft Piers

Park Road 22 (Proposed Bridge Borings)

Corpus Christi, Texas

East Abutment (Boring B-1)

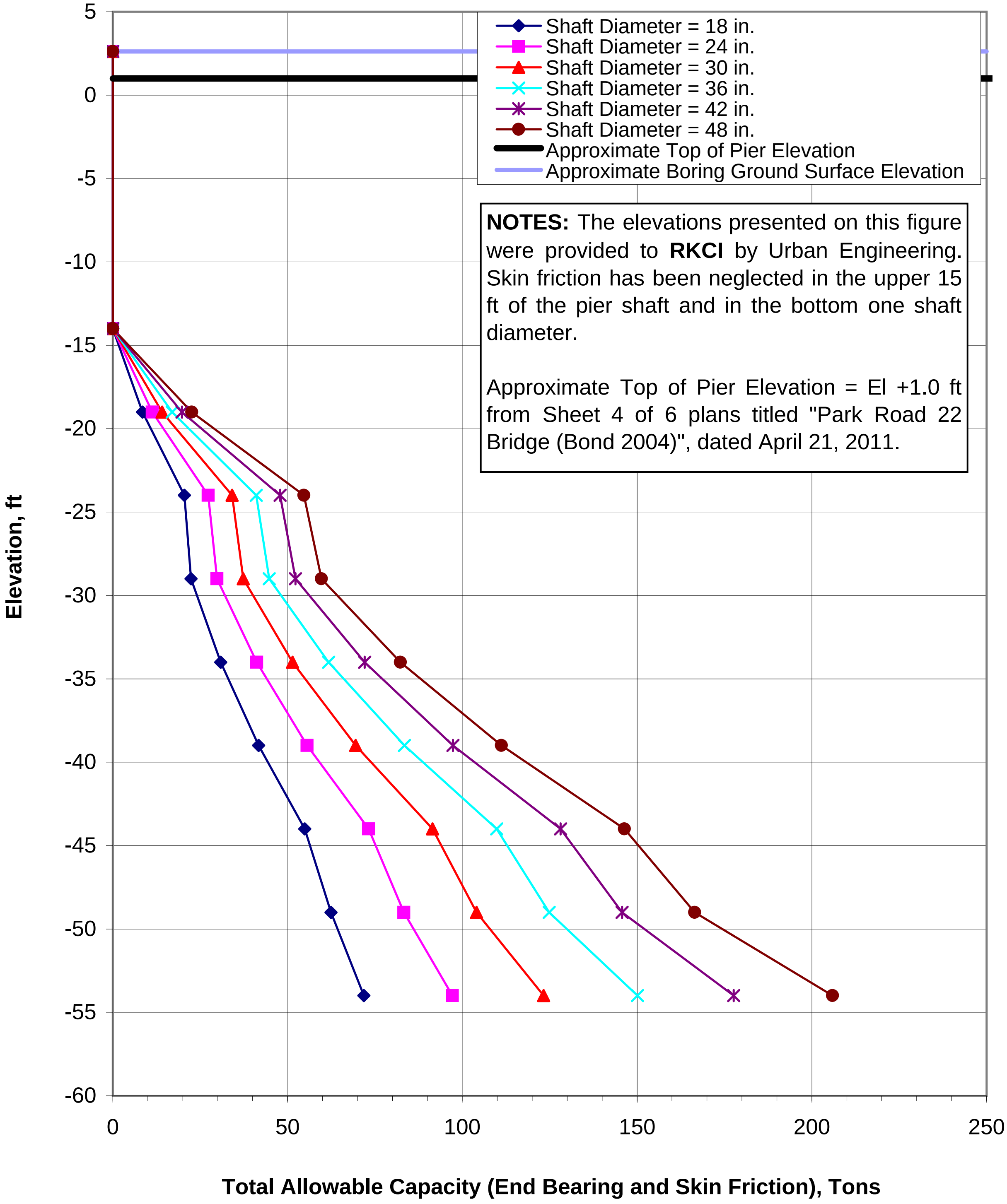


NOTES: The elevations presented on this figure were provided to **RKCI** by Urban Engineering. Skin friction has been neglected in the upper 15 ft of the pier shaft.

Approximate Top of Pier Elevation = El +1.0 ft from Sheet 4 of 6 plans titled "Park Road 22 Bridge (Bond 2004)", dated April 21, 2011.

DRILLED PIER AXIAL CAPACITY CURVE

Straight Shaft Piers
Park Road 22 (Proposed Bridge Borings)
Corpus Christi, Texas
Bent (Boring B-2)



NOTES: The elevations presented on this figure were provided to **RKCI** by Urban Engineering. Skin friction has been neglected in the upper 15 ft of the pier shaft and in the bottom one shaft diameter.

Approximate Top of Pier Elevation = El +1.0 ft from Sheet 4 of 6 plans titled "Park Road 22 Bridge (Bond 2004)", dated April 21, 2011.

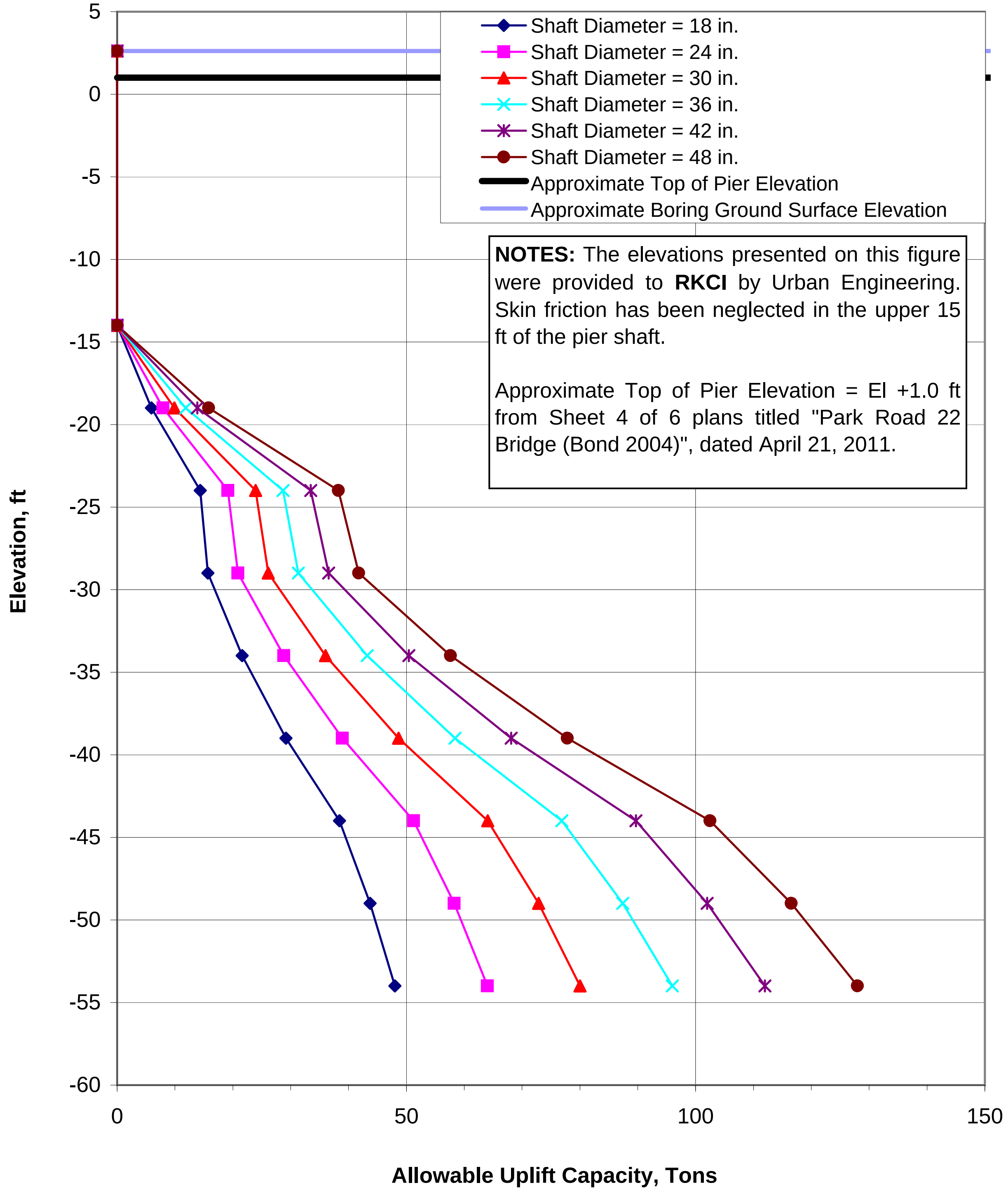
DRILLED PIER UPLIFT CAPACITY CURVE

Straight Shaft Piers

Park Road 22 (Proposed Bridge Borings)

Corpus Christi, Texas

Bent (Boring B-2)

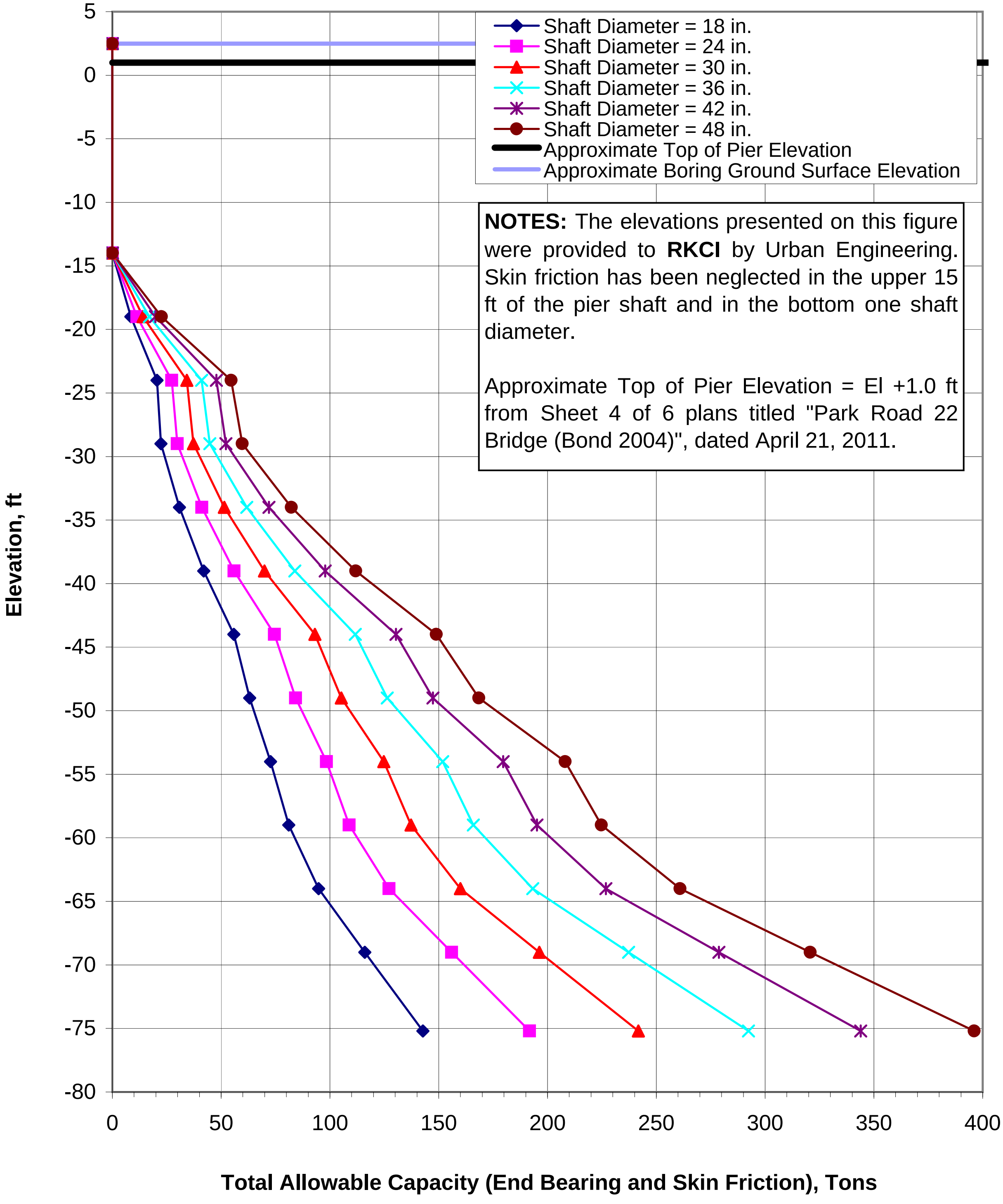


NOTES: The elevations presented on this figure were provided to **RKCI** by Urban Engineering. Skin friction has been neglected in the upper 15 ft of the pier shaft.

Approximate Top of Pier Elevation = El +1.0 ft from Sheet 4 of 6 plans titled "Park Road 22 Bridge (Bond 2004)", dated April 21, 2011.

DRILLED PIER AXIAL CAPACITY CURVE

Straight Shaft Piers
Park Road 22 (Proposed Bridge Borings)
Corpus Christi, Texas
Bent (Boring B-3)



NOTES: The elevations presented on this figure were provided to **RKCI** by Urban Engineering. Skin friction has been neglected in the upper 15 ft of the pier shaft and in the bottom one shaft diameter.

Approximate Top of Pier Elevation = El +1.0 ft from Sheet 4 of 6 plans titled "Park Road 22 Bridge (Bond 2004)", dated April 21, 2011.

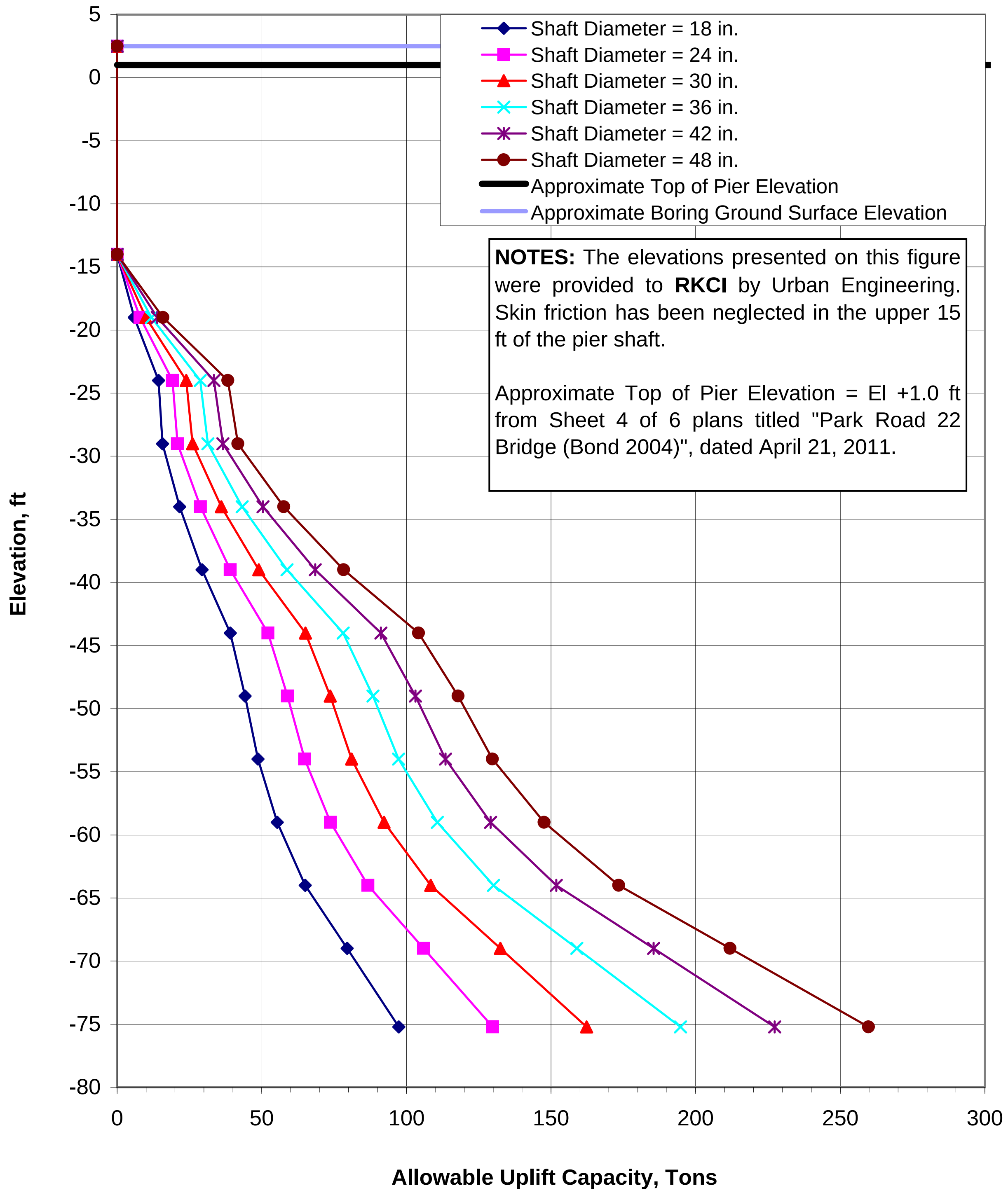
DRILLED PIER UPLIFT CAPACITY CURVE

Straight Shaft Piers

Park Road 22 (Proposed Bridge Borings)

Corpus Christi, Texas

Bent (Boring B-3)



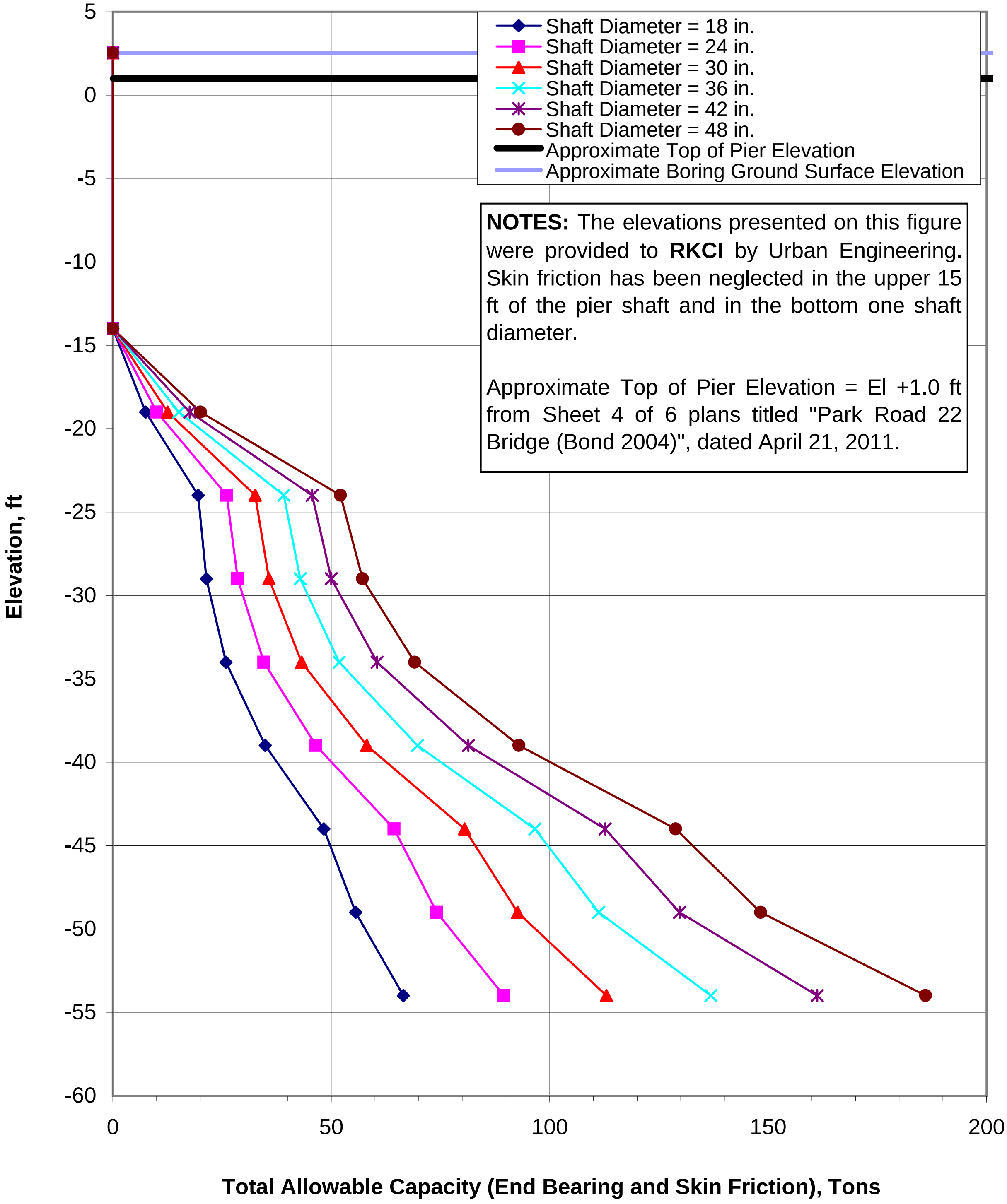
NOTES: The elevations presented on this figure were provided to **RKCI** by Urban Engineering. Skin friction has been neglected in the upper 15 ft of the pier shaft.

Approximate Top of Pier Elevation = El +1.0 ft from Sheet 4 of 6 plans titled "Park Road 22 Bridge (Bond 2004)", dated April 21, 2011.

DRILLED PIER AXIAL CAPACITY CURVE

Straight Shaft Piers
Park Road 22 (Proposed Bridge Borings)
Corpus Christi, Texas

West Abutment (Boring B-4)



NOTES: The elevations presented on this figure were provided to **RKCI** by Urban Engineering. Skin friction has been neglected in the upper 15 ft of the pier shaft and in the bottom one shaft diameter.

Approximate Top of Pier Elevation = El +1.0 ft from Sheet 4 of 6 plans titled "Park Road 22 Bridge (Bond 2004)", dated April 21, 2011.

Figure C-7

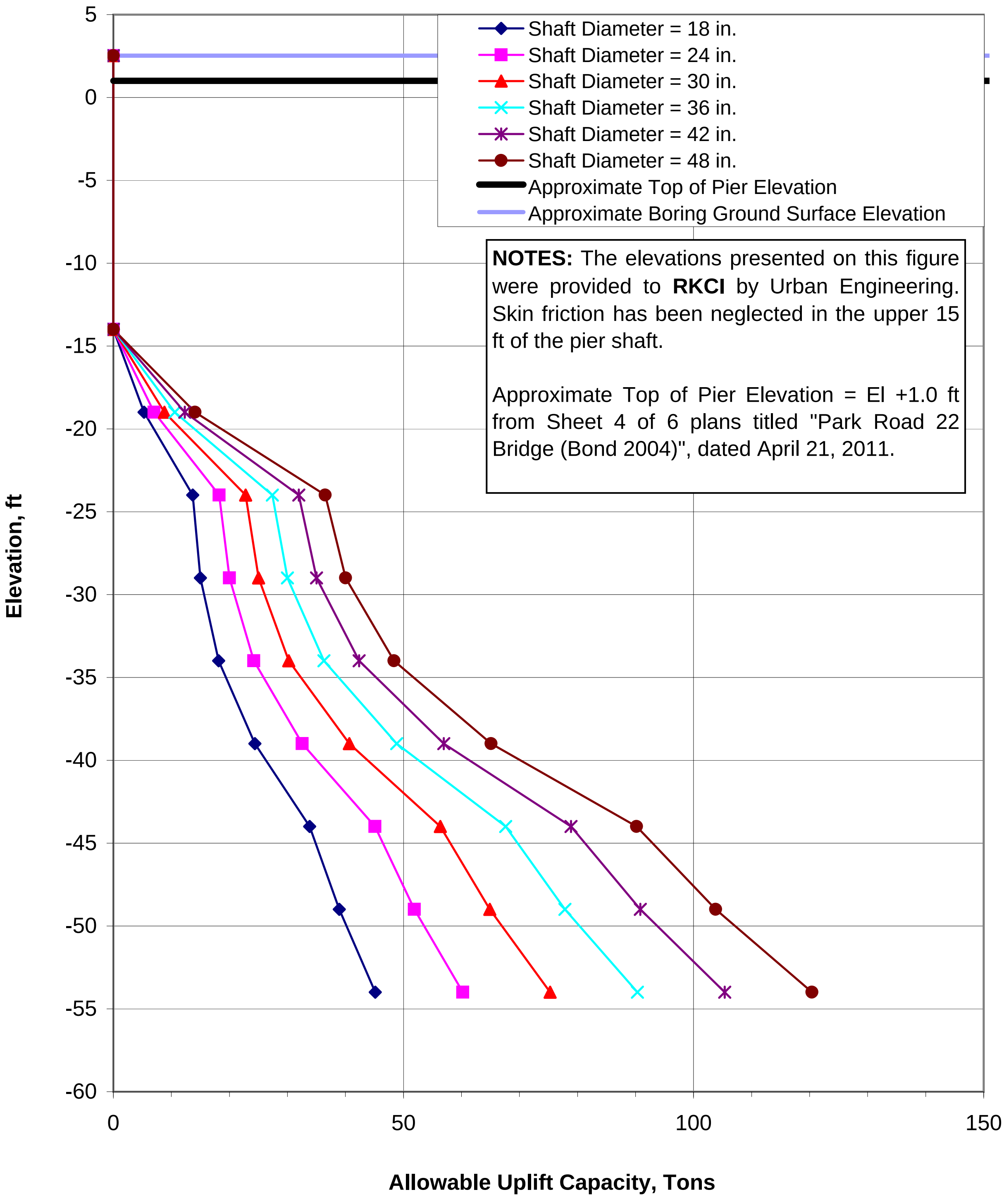
DRILLED PIER UPLIFT CAPACITY CURVE

Straight Shaft Piers

Park Road 22 (Proposed Bridge Borings)

Corpus Christi, Texas

West Abutment (Boring B-4)



NOTES: The elevations presented on this figure were provided to **RKCI** by Urban Engineering. Skin friction has been neglected in the upper 15 ft of the pier shaft.

Approximate Top of Pier Elevation = El +1.0 ft from Sheet 4 of 6 plans titled "Park Road 22 Bridge (Bond 2004)", dated April 21, 2011.

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you —* should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

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